Key Note Address

The Storebælt Link - a geotechnical view

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Published in:
The Interplay between Geotechnical Engineering and Engineering Geology : Proceedings of the Eleventh European Conference on Soil Mechanics and Foundation Engineering, Copenhagen, 28 May - 1 June 1995 : XI ECSMFE E

Publication date:
1995

Document Version
Publisher's PDF, also known as Version of record

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The Storebælt Link - a geotechnical view

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Synopsis: The 18 km fixed link across Storebælt will be the first of three major infrastructure projects - Storebælt Link, Øresund Link and Femer Bælt Link - linking Denmark internally and externally to her neighbours. It is one of the major per capita investment civil engineering projects today and includes a world record spanning suspension bridge, the most extensive precasting operation for a major bridge with elements weighing up to 7400 tons and a tunnel driven through probably the most difficult soil conditions ever encountered with full face tunnel boring machines.
The paper presents an overview of the Storebælt Link project from a geotechnical stand point. The extent, complexity and challenges this offered to the geotechnical profession are demonstrated by two specific examples. Finally, the spin-off, innovations and lessons learned are briefly described.

1. INTRODUCTION

It was with some reservation that the first author accepted the challenge from the Scientific Committee to present the first Key Note Lecture to the 11th European Conference on Soil Mechanics and Foundation Engineering.

The apprehension was due to my formal status in the Conference Organisation. However, the topic for the lecture - the Storebælt Link Project - was the very reason I left academia to re-join the Danish Geotechnical Institute in order to get involved in this exciting project. Thus, I am also very pleased to be able to present a geotechnical view of the Storebælt Link Project.

It goes without saying that a large number of companies and individuals have participated in the project - even in the field of geotechnical engineering. The account will therefore not be the objective, true story, but coloured by the engagement of my co-author and myself with the Danish Geotechnical Institute as consultants to the owner, A/S Storebæltsforbindelsen.

2. HISTORY OF STOREBÆLT LINK

2.1 *Why a link?*

Storebælt (cf Figure 1) is one of the important internal and international waterways splitting the Danish Kingdom in two parts.

A fixed link has been up for debate for as long as the technology was believed to be at hand. Only occasionally has Nature provided a fixed link - during severe winters.

King Carl X Gustav of Sweden took advantage of such a link when he in 1658 was presented with a declaration of war by the Danish king during a warring campaign in Poland.

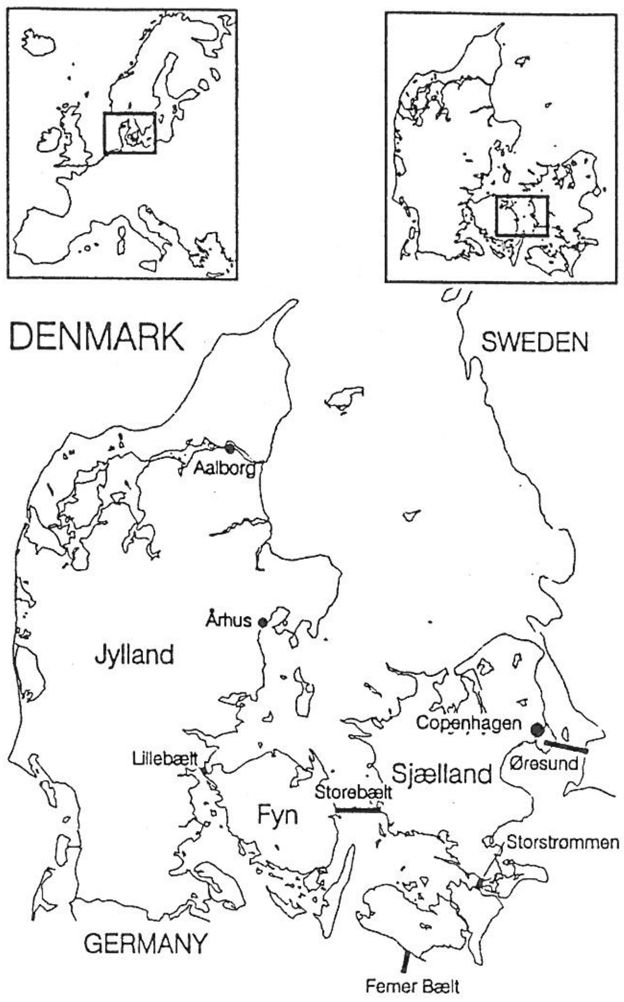


Fig. 1. Map of Denmark with Storebælt 's geographical position

He force marched his army through Germany to Jylland and crossed Lillebælt and Storebælt, both frozen solid, to defeat the Danish army on Sjælland. As a result the Danish provinces across Øresund were surrendered and, hence, the current Øresund Link project is being completed bi-laterally with our now friendly Swedish neighbours.

The rationale for realising the project has varied over the past two Centuries as has the reasons for opposing it.

Pros for establishment of Link:

- Storebælt splits the five million Danish population in two equal parts
- Replacing the current ferry traffic with a fixed link will halve the travelling time between Copenhagen and Århus (the two biggest cities) from 5 to 2.5 hours
- Expected daily traffic volume of 26,000 train passengers and 16,000 cars (ie a doubling of the current volume)
- The Link will be part of the European infrastructure projects

The arguments for the Link have changed slightly in time as shown in Table 1.

Table 1. Arguments for a Link since 1848

Year	Argument
	Military strategic reasons
1855	Extension of Danish railway system to Fyn
	Loss by farmers and retailers during ice winters
1892-93	equal to the cost of a tunnel
1940's	3 ice winters ≈ cost of Link
	Relief of unemployment
1908	Construction to employ 1500 men in several years
1936	Estimate of 100 M DKK/year in saved unemployment benefits
1950	An estimated work force of 12,000 men with financing by the American Marshall aid
	Relief of traffic jams
1950	Easter traffic up by 25% and 4 km car queues in Korsør
1955	10 km car queues during holidays

Cons for establishment of Link:

- Environmental considerations (after 1970)
- Lack of profitableness
- Renunciation of the "famed" Storebælt coffee on the ferries (23 and 24 December 1953: 22,000 cups and 21,000 *Danish*)

That the project alters the geography - for better and for worse - is clear from Figure 2 depicting Sprogø, the small island in the middle of Storebælt. Through land reclamation to accommodate the tunnel ramp area and the landfalls of the bridges and sedimentation basins the original island has grown to more than 3 times the original size.

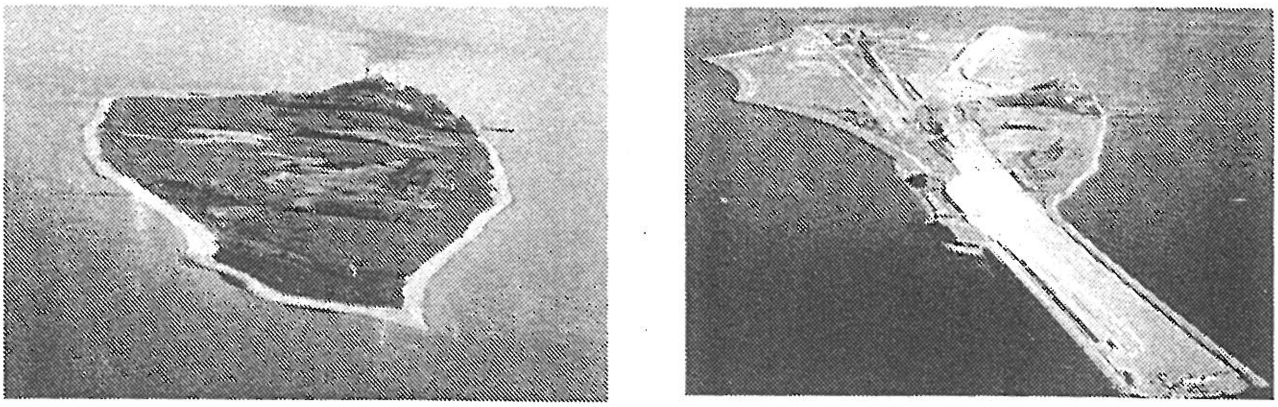


Fig. 2. Bird's view of Sprogø (in the middle of Storebælt) before and after commencement of the Storebælt Link

2.2 *Chronology*

Table 2 lists some of the milestones in the establishment of the Storebælt Link. Most of the historical information on Storebælt has been drawn from a compilation by Møller & Wichmann (1978).

Table 2. Milestones in the chronology of the Storebælt Link

Time	Milestone
1855	Tunnel proposal by War Minister Tscherning
1908	First concrete proposal for link as a tunnel by H. Ohrt
1934	Bridge proposal by Hiort-Lorentzen
1936	Bridge proposal by Danish/Swedish contractors
1962-63	Soil investigations for feasibility study
1965/66	International design competition for Storebælt Bridge
1968	Final recommendation from the Storebælts-commission after 11 years of deliberations
1977-78	Soil investigations for first attempt at link project - abandoned politically
1983	Soil investigations for bored tunnel
1986-98	Realisation of current project

In a debate in the Danish Parliament, War Minister Tscherning in 1855 argued that an extension of the Danish railway system outside Sjælland (the island with the Capital) would be a folly as long as a fixed link across Storebælt was not established. He was imagining a tunnel under the Belt similar to a proposal for the English Channel from 1802, as shown in Figure 3.

The crossing was envisioned by carriage and four with the tunnel lit by thousands of candles.

Despite his warnings the railway system was extended with crossing of Storebælt ensured by ferries.

The first concrete proposal for a link (including a tentative soil profile) - a tunnel - was proposed by a surveyor, H. Ohrt, in 1908. It was prompted by yet another severe spell of harsh winter weather of 1892-93. One of his concerns was to establish an unhindered passage from the Capital to England (via Esbjerg).

In 1930 the Danish Railways launched a car ferry despite some serious doubts on profitableness. However, the traffic volume increased and in 1934, while fixed links (bridges) were being constructed across two other belts, Lillebælt and Storstrømmen (cf Figure 1), the retired Chief Engineer of DSB, R. Hiort-Lorentzen, proposed a bridge across Storebælt.

He argued that (his estimate of) 40 DKK per capita was a low price to pay to provide a fixed connection between the 1.9 million people East of the Belt and the remaining 1.6 million people of Denmark, which, he pointed out, was the only country with her Capital on an island.

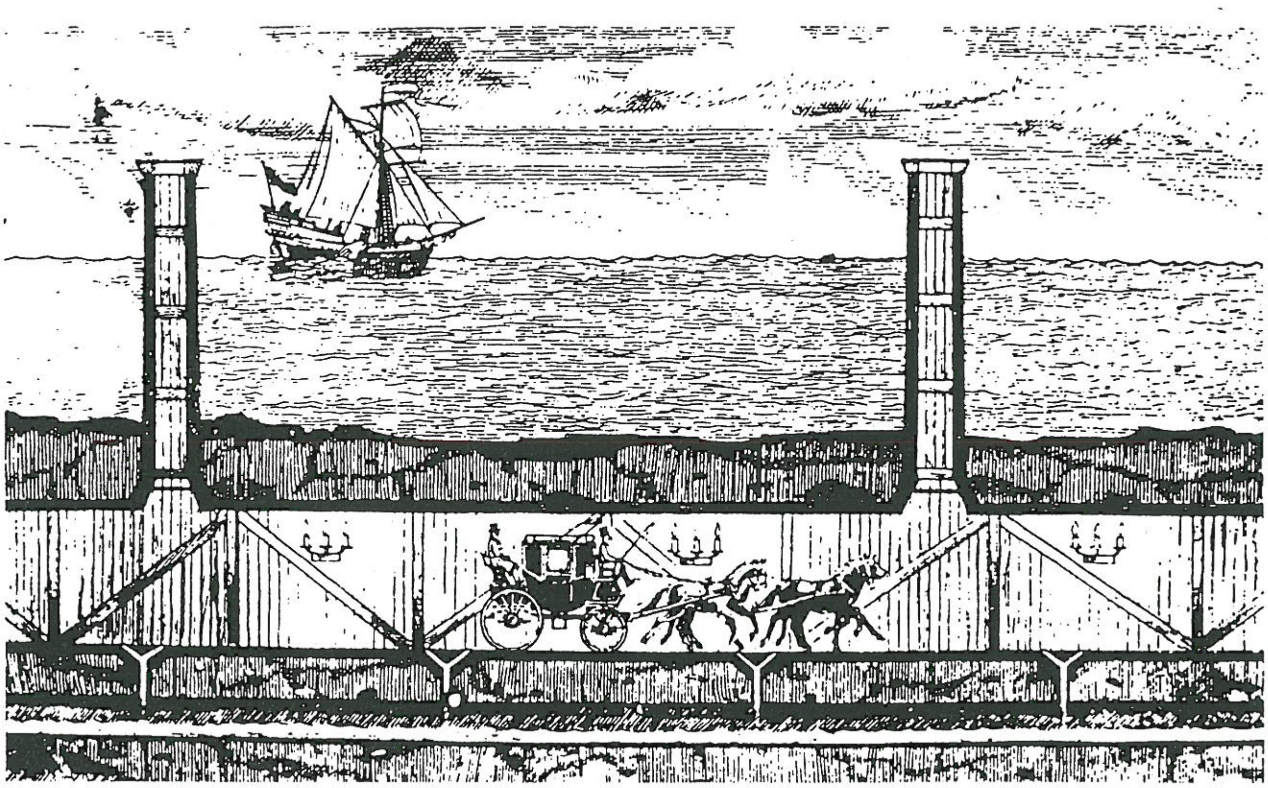


Fig. 3. Proposal for tunnel under the English Channel from 1908

To put the costs in perspective he drew attention to the ongoing construction of two bridges in the San Francisco Bay area which would cost her inhabitants 340 DKK each.

In 1936 three major Danish contractors (together with two Swedish contractors for the Øresund link) published a White book on the future expansion of the Danish motor way system including fixed links across Storebælt and Øresund in the shape of bridges. The proposal was much acclaimed in the press.

However, it became blatantly clear already at this time, that the fixed link would be subject to endless debates and disputes over costs and necessity. A large number of commissions were established and abandoned over the years and "recommendation" no 508 saw the light of day in 1968! This prompted one of the Danish newspaper cartoonists to offer his "clear picture" of the bridge, as depicted in Figure 4 (*"Committee after committee - debate after debate have contributed to give a clear picture of the layout of the bridge"*).

Although the Link was almost synonymous with a bridge from the 1930's an onwards, a number of other proposals also saw the light of day. There was even talk of flying the cars across Storebælt to do something about the increasing holiday queues of several kilometres of cars on either side of the Belt.

The debate prompted the proposal in Figure 5 from one of the newspapers. *The "bridge" is disrupted at Sprogø by a tunnel which passes under a channel allowing unhindered passage by shipping and maintaining Storebælt as an international water way!*

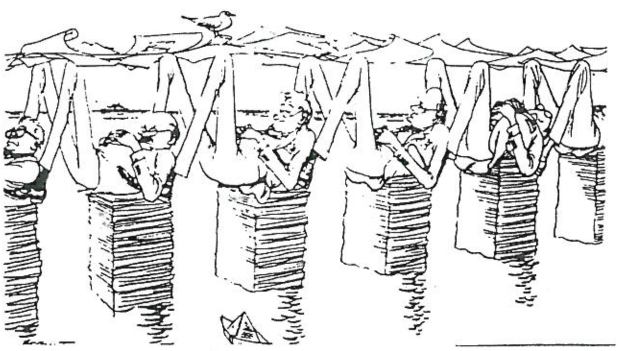


Fig. 4. Self-supporting construction. (after Bo Bojesen 1958).

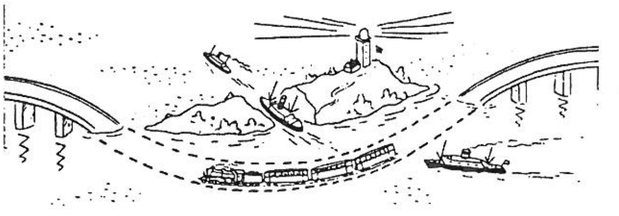


Fig. 5. A newspaper "proposal" for the layout of the Link in 1952

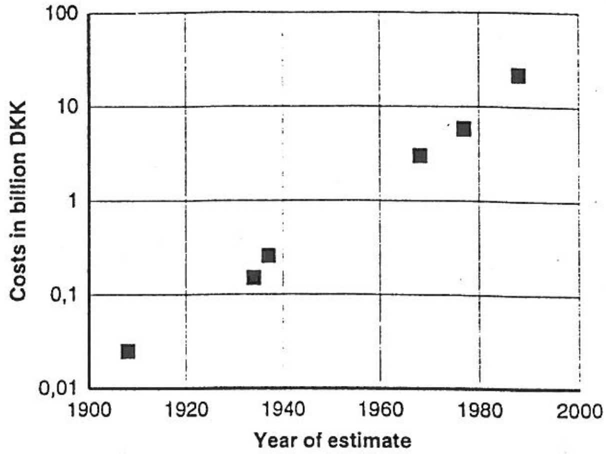


Fig. 6. Estimated costs of the Link versus time of proposal

2.3 *Costs*

The estimated costs of a link has been controversial to this day. Figure 6 shows the escalation of the cost estimates from 25 million DKK in 1908 to the 21.6 billion DKK for the current project (in 1988 prices). As the rise is logarithmic it would appear that the sooner we finish the project the better.

The owner and client for the current project is the Danish limited liability company A/S Storebæltsforbindelsen with the Danish State as sole shareholder. The company undertakes the design, construction and operation of the Fixed Link.

To finance the project A/S Storebæltsforbindelsen raises loans in Denmark and abroad. The project budget is DKK 21.6 billion in January 1988 price level. Repayment of the loans will commence on completion of the link, when the company starts receiving toll charges for the use of the link. The Danish State Railways will pay a substantial fee for the use of the link.

3. COMPONENTS OF LINK PROJECT

Figure 7 shows the three main components of the Storebælt Link:

- The West Bridge - the longest combined railway and motor way bridge in Europe,
- the East tunnel - the second longest tunnel under water in Europe, and
- the East Bridge - a world record suspension bridge with a main span of 1624 m.

In the following a brief presentation of these components are presented together with the foundation solutions selected.

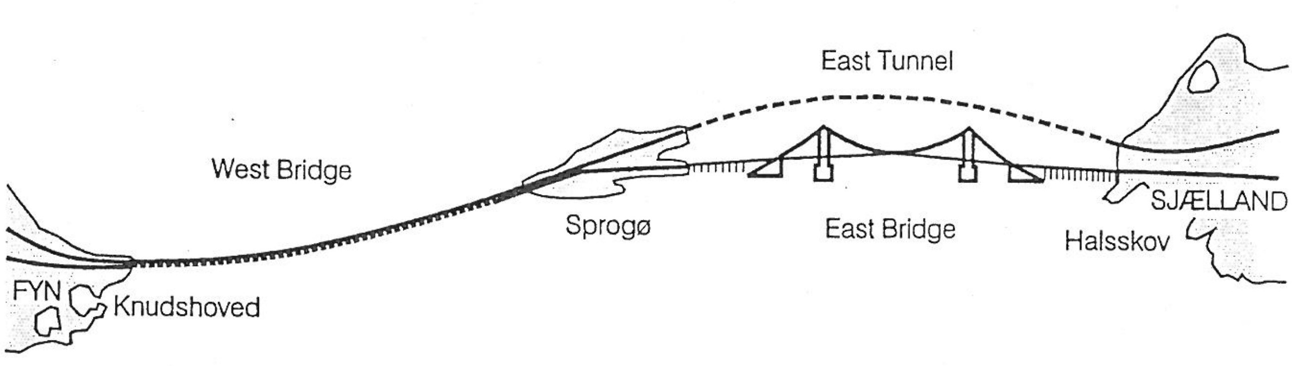


Fig. 7. Main components of the Storebælt Link between Knudshoved on Fyn and Halsskov on Sjælland

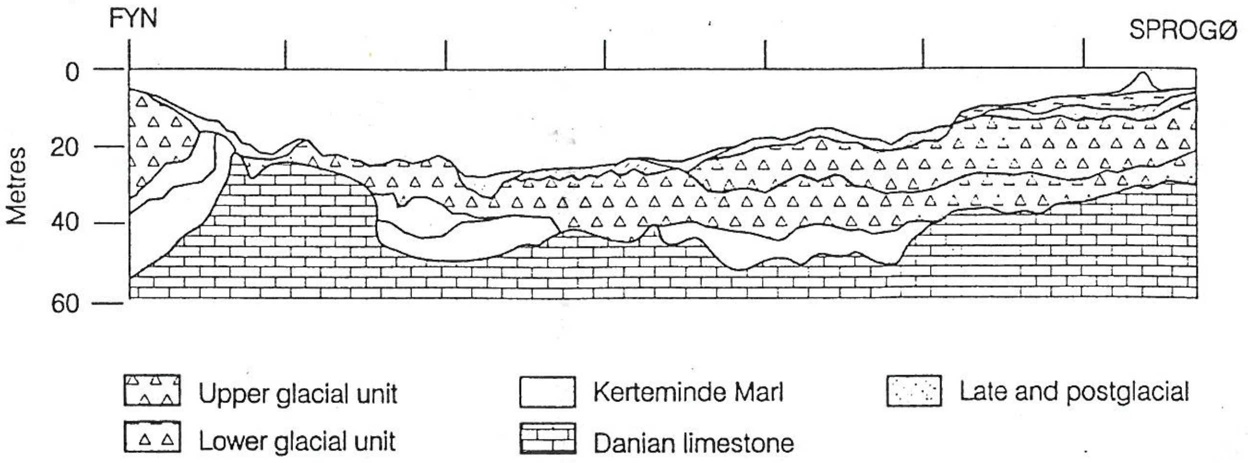


Fig. 8. Schematic geological cross section of Storebælt shown along the alignments of (to the left)

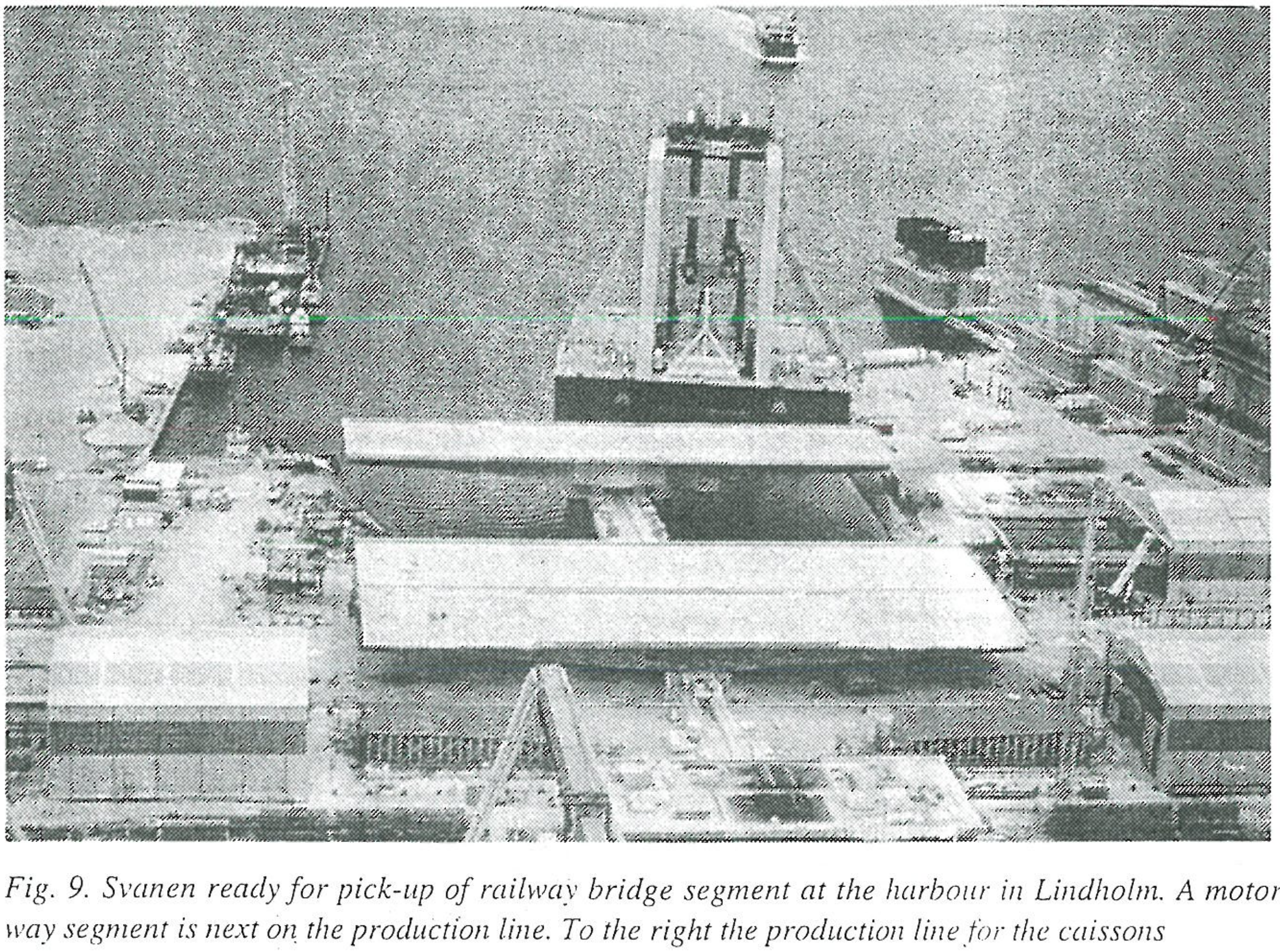
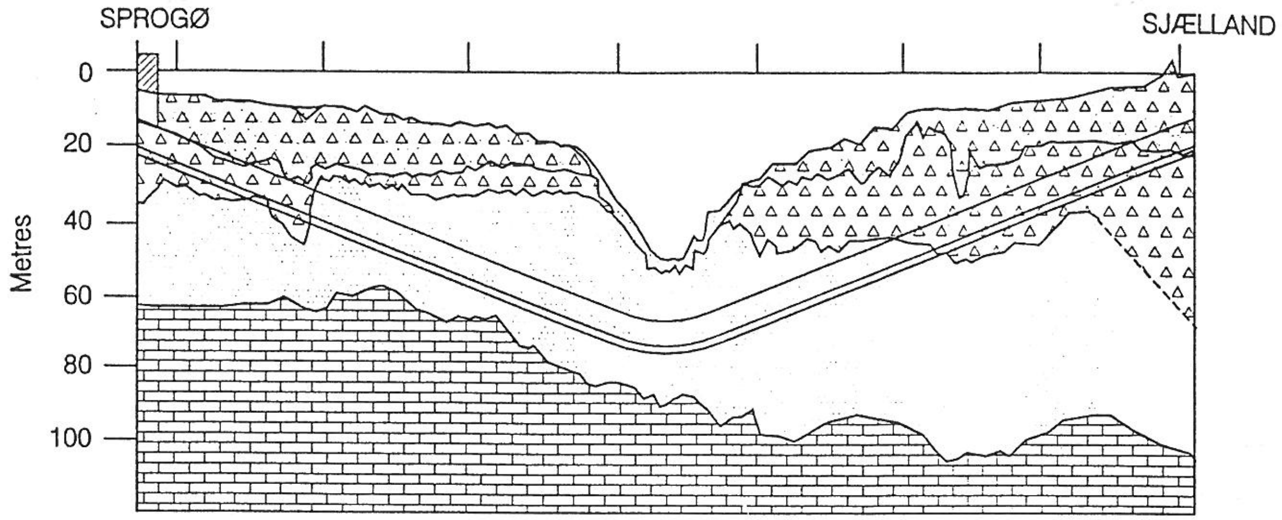


Fig. 9. Svanen ready for pick-up of railway bridge segment at the harbour in Lindholm. A motor way segment is next on the production line. To the right the production line for the caissons

3.1 *Overall geology*

Through the different site investigation campaigns the overall geological setting of the Storebælt area is now well understood (cf Foged et al 1995; Larsen et al 1982). A simplified, schematic section following the West Bridge and East tunnel alignments is shown in Figure 8 in order to give an overall understanding of the foundation solutions chosen for the three main components of the Link.



the West Bridge, and (to the right) the East tunnel (after Foged et al, 1995)

The Glacial Till complex and the underlying Selandian *Kerteminde Marl* are the two key soil strata for the foundation solutions.

3.2 *West Bridge*

The West Bridge was constructed by ESG - European Storebælt Group consisting of:

- ▼ Højgaard & Schultz A/S — Denmark
- ▼ Ballast Nedam Civil Engineering — The Netherlands
- ▼ Taylor Woodrow Construction Ltd. — United Kingdom
- ▼ Losinger Ltd. Contractors & Civil Engineers — Switzerland
- ▼ C.G. Jensen A/S — Denmark
- ▼ Per Aarsleff A/S — Denmark

The construction period was June 1989 to January 1994. The completed bridge is seen in Figure 14.

The tender design for the West Bridge was prepared by CCL consisting of COWI, Carl Bro Group and Leonhart Andrä und Partner.

The West Bridge is a 6.6 km long low bridge with concrete sub and super structure. All elements of the bridge were prefabricated at an onshore element casting yard in Lindholm near the landfall on Fyn. The plant facility had its own harbour, facilitating pick-up of elements by the giant crane Svanen as shown in Figure 9. It is the most extensive pre-casting operation for a major off-shore bridge project carried out to date.

A cross section of the bridge is shown in Figure 10. The railway and motor way have separate pier shafts resting on a common sand-filled caisson placed in an excavation with a screeded, crushed stone layer.

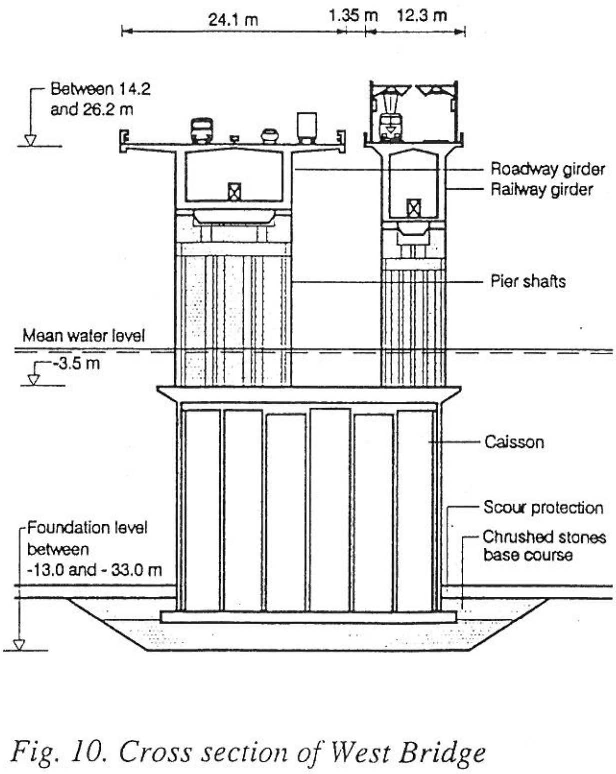


Fig. 10. Cross section of West Bridge

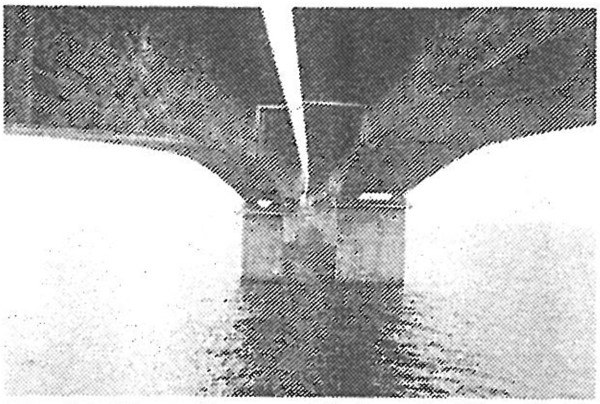


Fig. 11. View along and below the West Bridge alignment

Fig. 12. View inside a West Bridge motor way segment

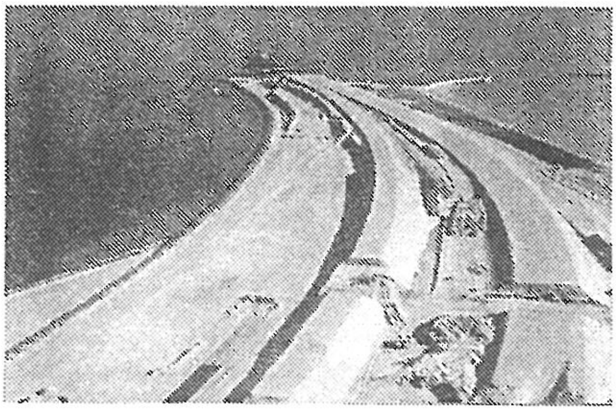


Fig. 13. Approach ramps on Fyn to West Bridge railway and motor way

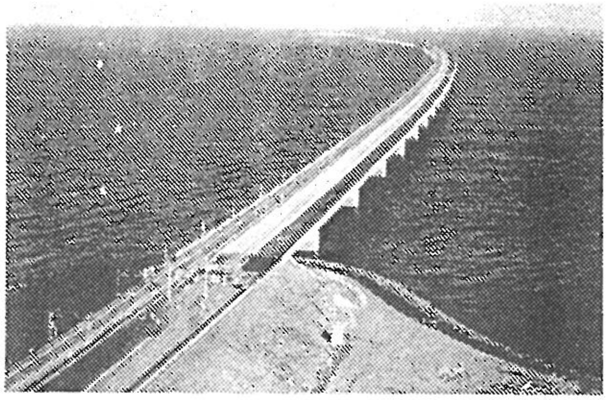


Fig. 14. View of completed West Bridge, 1995, from Fyn

The maximum foundation depth was determined by the 6700 tons lifting capacity of *Svanen*. However, aided by submersion the maximum element size was 7400 tons.

Figures 11 and 12 show views from below and inside the super structure of the West Bridge. Walking the length of the Bridge internally gives a certain "Cathedral" feeling.

The division of railway and motor way is also very clear from the construction of the approach ramps in Figure 13. At the end of the ramps excavation and placing of crushed stone bed are on-going from a jack-up.

The completed West bridge, seen from Fyn, is shown in Figure 14.

The foundation details and the behaviour of the completed bridge are described by Kristensen et al (1995).

3.3 *East tunnel*

The contractor for the East Tunnel is the joint venture MT Group - MTG consisting of:

▼ Monberg & Thorsen A/S	Denmark
▼ Campenon Bernard	France
▼ SOGEA	France
▼ Dyckerhoff & Widmann	Germany
▼ Kiewit Construction Company	USA

The designers were the COWI-Mott Joint Venture consisting of COWI and Mott McDonald .

Due to a number of incidents the work on the tunnel has suffered delays. Construction started in November 1988 and is expected to be completed in 1996 for opening of traffic in 1997.

The East Tunnel is a 7.4 km long twin tubed, bored tunnel, as seen in Figure 15. It passes through varying till deposits and marl and has a Nadir point at level -78 m. The soil cover varies from ≈ 15 m over the Nadir point,

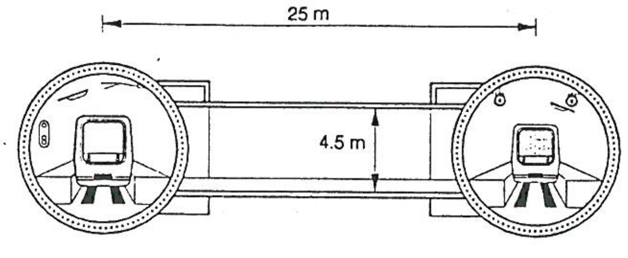


Fig. 15 Cross section of East Tunnel

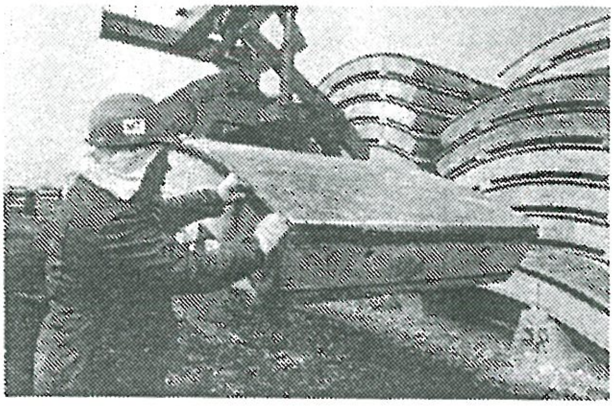


Fig. 16 Storage of concrete lining segments

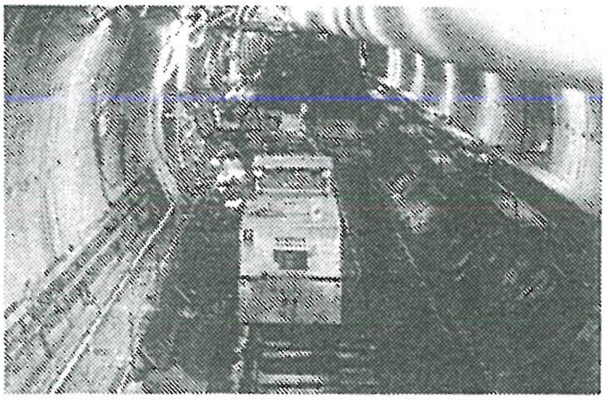


Fig. 17 View of main tube during construction

where the water depth is > 50 m, to some 40 m at the most.

Four full face TBMs with an 8.7 m diameter bore were employed to complete the two main tunnel tubes. These are lined with 63,000 pre-cast concrete lining segments 0.4 m thick.

The segments were produced at a purpose-built element plant in Halsskov. Production went very smoothly, and the only major problem turned out to be storage room due to the delay in the mining process (see Figure 16).

Segments were sailed to Sprogø to feed the two TBMs working from this side of the tunnel.

A view into the main tunnel during construction is seen in Figure 17. Each tunnel ring consists of seven concrete segments, six of them weighing 7 tonnes, one weighing two tonnes.

Between the two main tubes, 29 cross passages were subsequently mined "by hand" to provide space for electric installations and emergency exits. The cross passages are lined with cast iron segments.

Two further cross passages are positioned in the two 300 m long cut-and-cover areas at each end of the tunnel.

Figure 18 shows work in progress at the Sprogø cut-and-cover area. In-situ casting of the main tunnel tubes outside the starting chamber for the TBMs are ongoing at the back and casting of the portal building at the front.

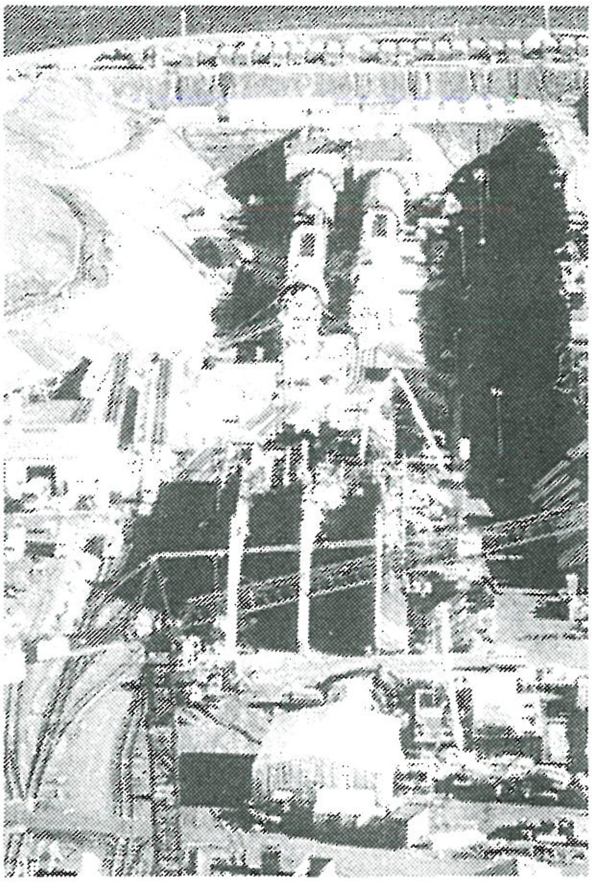


Fig. 18 Work in progress in the cut-and-cover area on Sprogø

3.4 *East Bridge*

Two consortia are responsible for the construction of the last leg of the Storebælt Link, the East Bridge.

The substructure is constructed by the Great Belt Contractors - GBC a joint venture of:

- ▼ Hochtief AG — Germany
- ▼ Wayss & Freytag AG — Germany
- ▼ HBW Hollandsche Beton- en Waterbouv b.v. — The Netherlands
- ▼ KKS entreprise A/S — Denmark
- ▼ E. Pihl & Søn A.S. — Denmark

The joint venture for the superstructure is EBC

- ▼ Iritecna-CFM sud
  East Bridge Consortium — Italy

in association with Steinman Boynton Gronquist & Birdsall, USA.

The design was performed by the joint venture CBR, consisting of COWI and Ramboll (formerly COWIconsult, B. Højlund Rasmussen and Rambøll & Hannemann).

The construction period is from August 1992 to 1998.

The East Bridge is an elevated 6.8 km long 4-lane motor way suspension bridge with a concrete substructure and a steel superstructure.

As shown in Figure 19 it has a world record main span of 1624 m, with a navigational clearance of 65 m. It is also apparent from Figure 19 that the geology is similar to that of the tunnel alignment (Figure 8b).

Following the experience from the West Bridge, direct foundation on compacted and screeded crushed stone beds was also chosen for the East Bridge anchor blocks, pylons and approach piers (cf Sørensen et al 1995).

The caissons for the piers, pylons and anchor blocks were precast in dry-docks in Kalundborg (cf Figures 20, 21) some 40 km north of the bridge alignment.

The towing operations required six tugboats, but apart from a single weather lay over all operations were completely successful with high precision positioning of caissons.

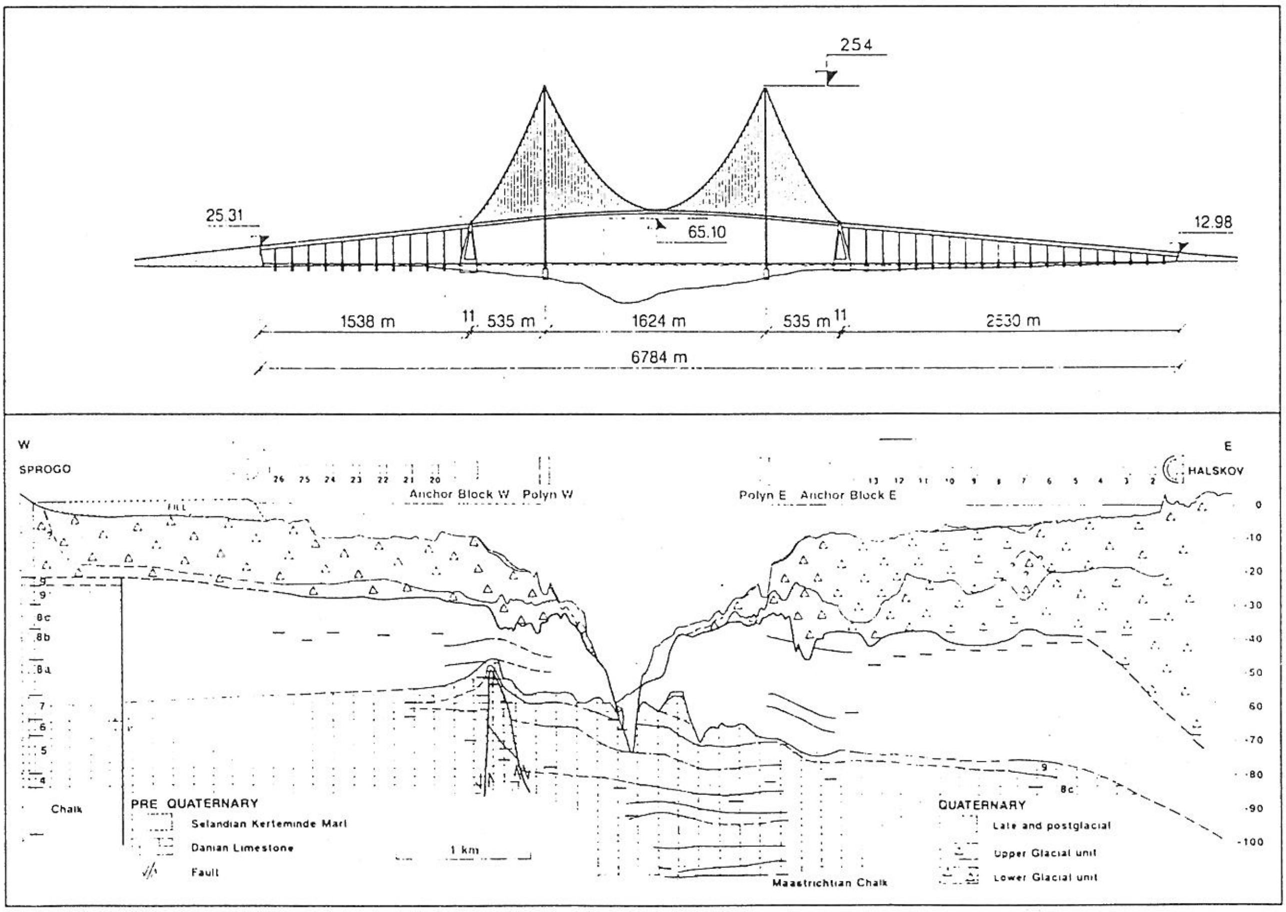


Fig. 19 Cross section along East Bridge alignment (Geological section after Foged et al 1995)

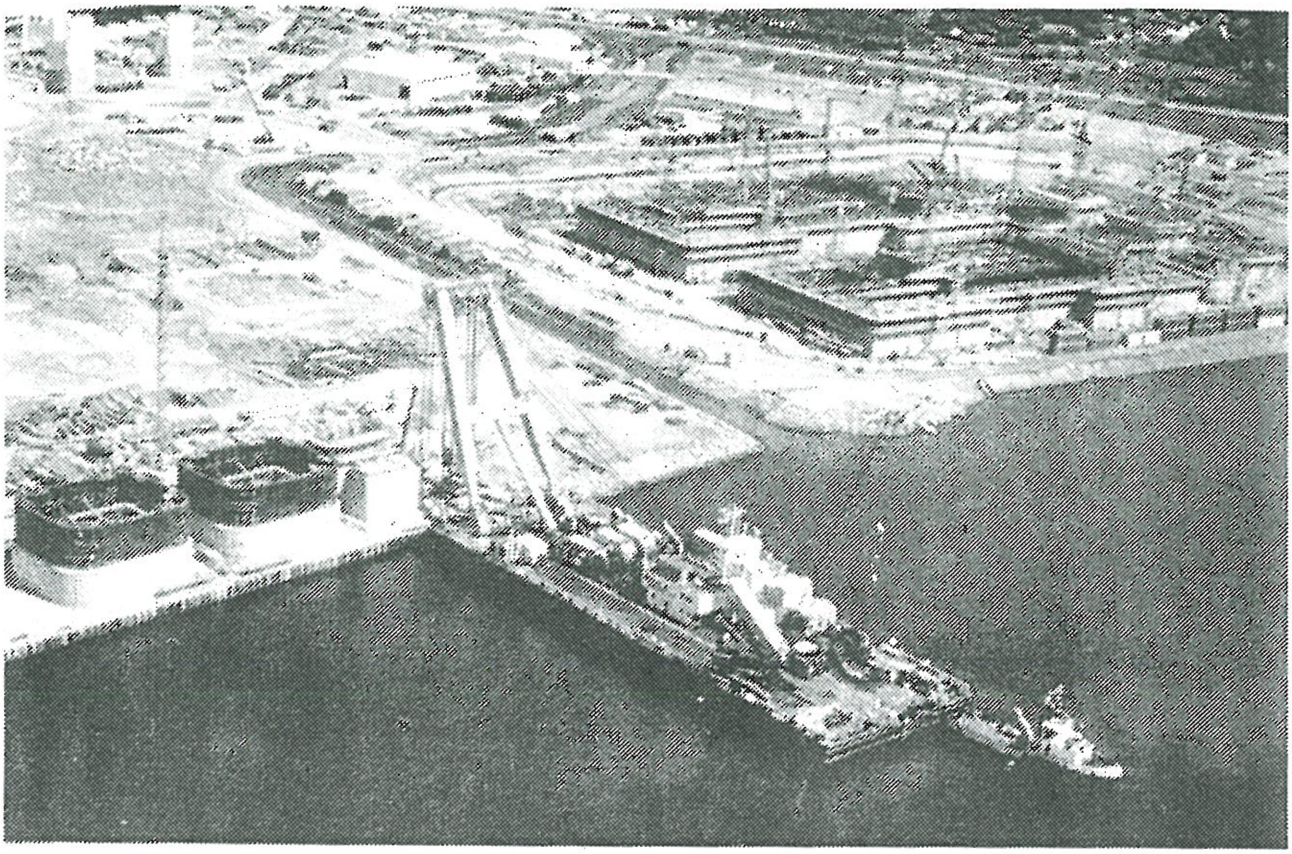


Fig. 20. Casting and dry-dock facilities in Kalundborg for the East Bridge. Anchor block caissons to the right and approach pier caissons to the left with barge and crane facility for transport

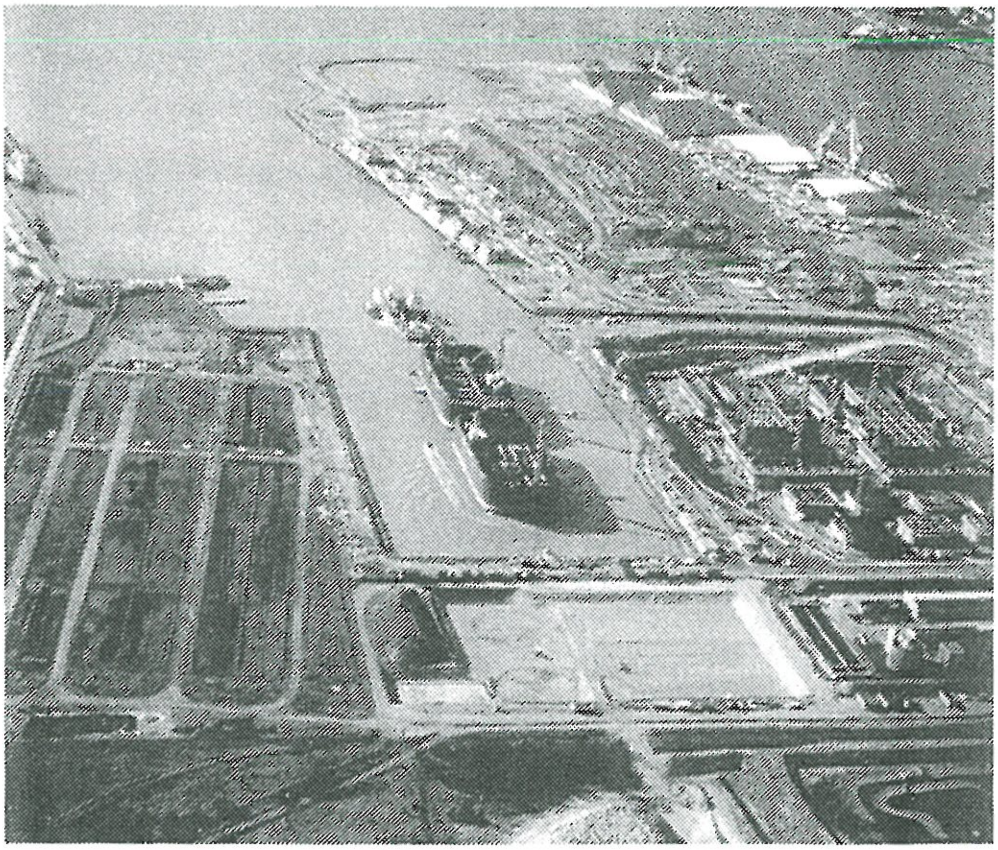


Fig. 21. Casting and dry-dock facilities in Kalundborg for caissons to the East Bridge. Towing operation of pylon caissons in progress

4. GEOTECHNICAL INVESTIGATIONS

4.1 *Strategy*

The underlying strategy of the geotechnical investigations for the Storebælt Link project aimed at:

- understanding the geological setting
- unravelling the depositional history
- establishing a viable and robust geological model for the alignment area
- understanding the link between geology and engineering
- quantifying the established model by field and laboratory testing.

It became a particular challenge to utilise the older investigations and to establish an electronic means of storing, searching, interchanging and accessing the diversified information from the geotechnical investigations by means of the so-called *Geomodel* (Porsvig & Christensen, 1994).

4.2 *Perspective*

Since the first geological model was presented by Ohrt in his 1908 proposal the focus, extent and importance of the geotechnical investigations have changed considerably.

To place the present knowledge in perspective, let us examine the state of knowledge at four different times:

- Ohrt, 1908
- Hiort-Lorentzen, 1934
- Christiani & Nielsen et al, 1936
- Foged et al 1995.

It is still surprising how Ohrt obtained his data as he presented a complete model with main boundaries and soil deposits as indicated in Table 3. Since the only deep boreholes (a total of three to 100 to 140 m depths) in the area were drilled as late as 1983, it is amazing that he was able to get as close as he did (compare with Figure 8). In presenting his proposal Ohrt ends with an often repeated request:

"Let us commence to carry out accurate, complete and reliable ground investigations."

Table 3. Ohrt's1908 geological model

Level [m]	Soil deposit to indicated level
-34	Clay till
-50	Dominantly sand and gravel
-94	Low permeable stony and shaly marl
-104	Limestone with flint

In response to the Hiort-Lorentzen's link proposal of 1934 the acting Chief Flensborg of the Danish State Railways (DSB), *the leading executive officer in the art of civil engineering* stated:

"Investigations have never been carried out by responsible experts within the DSB. Before even contemplating to address such a notion (to establish a Link) the soil conditions must be investigated. To this date such investigations have not been considered".

Thus, the need for soil investigations were recognised - it was rather a question of who should do it.

The approach by the contractors in their 1936 proposal was, however, quite different. *"As the cost of the foundation of the piers is a relatively small part compared to the total costs for the project, they have been conservatively priced and one can feel confident against additional costs even if closer investigation should give rise to changes in the envisaged constructions."* This is probably a very early notion of the Eurocode term "the observational approach" (f.inst. EN 1997, 1995).

In their summary of the geotechnical investigations actually carried out for the Storebælt Link, Foged et al (1995) conclude that *"the geological and geotechnical investigations of Storebœlt is an example of an integrated use of advanced methods within our profession in combination with use of the data base Geomodel for a complicated site."*

The total costs of the geotechnical investigations and verifications amount to about DKK 0.5 billion, or 2.5% of the total costs of the realised project.

4.3 *Chronology and extent of investigations*

The geotechnical investigations followed the traditional segmentation and time progression as shown below, but with an overriding influence in daily work from the interplay and co-operation between engineers and geologists. The investigations comprised:

▼ Conceptual design requirements, covering
- Establishment of overall geological model
- Feasibility of Fixed Link
- Foundation methods

▼ Detailed design investigations
- Alignment optimisation
- Foundation design parameters
- Basis for tender documents

▼ Special investigations
- Weak spots
- Special features (geological, geotechnical, hydrological)
- Stress-strain-time influences on soil data

▼ Control investigations
- Verification of design parameters
- Verification of foundation levels
- Monitoring

Using the *Geomodel* the statistics on the extent of the site investigations may easily be examined. These consisted of borings, CPT tests, vibrocorings and gravity corings and they were assisted by the information obtained from the dewatering wells. Key figures are:

▼ Geotechnical borings
- a total of 2,227 borings, with 1,473 carried out offshore
- 25,533 m of retrieved samples

▼ CPT testing
- a total of 2,465 CPT tests where 2361 were carried out offshore
- a total of 25,590 m of soil penetration

▼ Dewatering wells
- a total of 329 wells with a total bore hole length of 10,804 m

Fig. 22. DGI mobile drilling rig at top of embankment for East Bridge approach ramp on Sprogø

The majority of the laboratory testing was carried out by DGI, as were the seismic investigations. This resulted in:

- more than 100 ton of sample material kept in storage
- Thousands of laboratory tests, ranging from classification tests to special tests in custom-made test set-ups,
- 3,329 km of seismic profiling

4.4 *Special features of investigations*

One of the special assignments is shown in Figure 22, where a mobile drilling rig is boring monitoring holes through the embankment for the approach ramp to the East Bridge on Sprogø (embankment crown 24 m above sea level). The borings are taken through the fill and into the underlying clay till and marl. Settlement gauges are placed in the fill, clay till and marl.

The stress increase below the embankment is of the same order of magnitude as the stress increase from pylons and anchor blocks for the East Bridge. Thus these borings provide a unique opportunity to obtain full scale data on the settlement and creep properties of both clay till and marl.

Apart from the special investigations, with development of purpose-built testing set-ups, the two most notable features were the introduction of modern seismic/geophysical investigations and CPT-testing.

Due to the long period of investigations (1962 - present) the Storebælt investigations reflect the changes in seismic profiling and interpretation from analogue to digital, from single to multi-channel and from poor to precise positioning. It also proved the value of geophysics as a tool for engineers (cf Hartwell et al, 1995) to the point where it today is an accepted and valued tool in practically all major geotechnical investigations in Denmark.

Before Storebælt, CPT was almost non-existing as a tool in Danish soil investigations. However, despite the complex ground conditions the Storebælt project triggered an extensive application of CPT testing. Notwithstanding the heterogeneous nature of the clay tills, with a high content of very coarse particles up to boulder size and highly varying undrained shear strength (from 50 to well over 1000 kPa), a total of more than 25 km of CPT probing were carried out in Storebælt.

This allowed a much more detailed and varied picture of the stratigraphy and the geotechnical parameters compared with traditional field vane shear testing. Simultaneously, this, however, also facilitated a very thorough calibration of CPT with the Danish "well-winnowed" experience from vane strength measurements (cf Mortensen et al, 1995).

Today CPT is considered a must in the in situ arsenal of testing tools for the Danish geotechnical profession.

However, the overriding message of the CPT experience from Storebælt was the need for interaction of testing tools (seismic profiling, borings, in situ and laboratory tests). Application of a single tool alone may seriously limit the possibilities of rendering a complete picture and achieve safe and economical solutions (see Steenfelt & Sørensen, 1995).

5. PROJECT MOSES

On the morning of October 14, 1991 a sorry sight met the eyes of spectators at the tunnel ramp area on Sprogø, as seen in Figure 23. The ramp area and the two Eastbound tunnel tubes were inundated in what is referred to as the *Jutlandia incident*.

During a period of maintenance stoppage of the TBM Jutlandia a chimney-like connection to Storebælt formed allowing ingress of sea water into the tunnel. Both TBMs and tunnel tubes were successfully evacuated without loss of lives, but both ongoing mining operations had to be postponed for a long time (restart of first eastbound TBM in August, 1992) to allow remediation and to ensure that a similar incident would not occur again at any of the four TBM-positions.

In order to control the mining operations project *MOSES* was set in action.

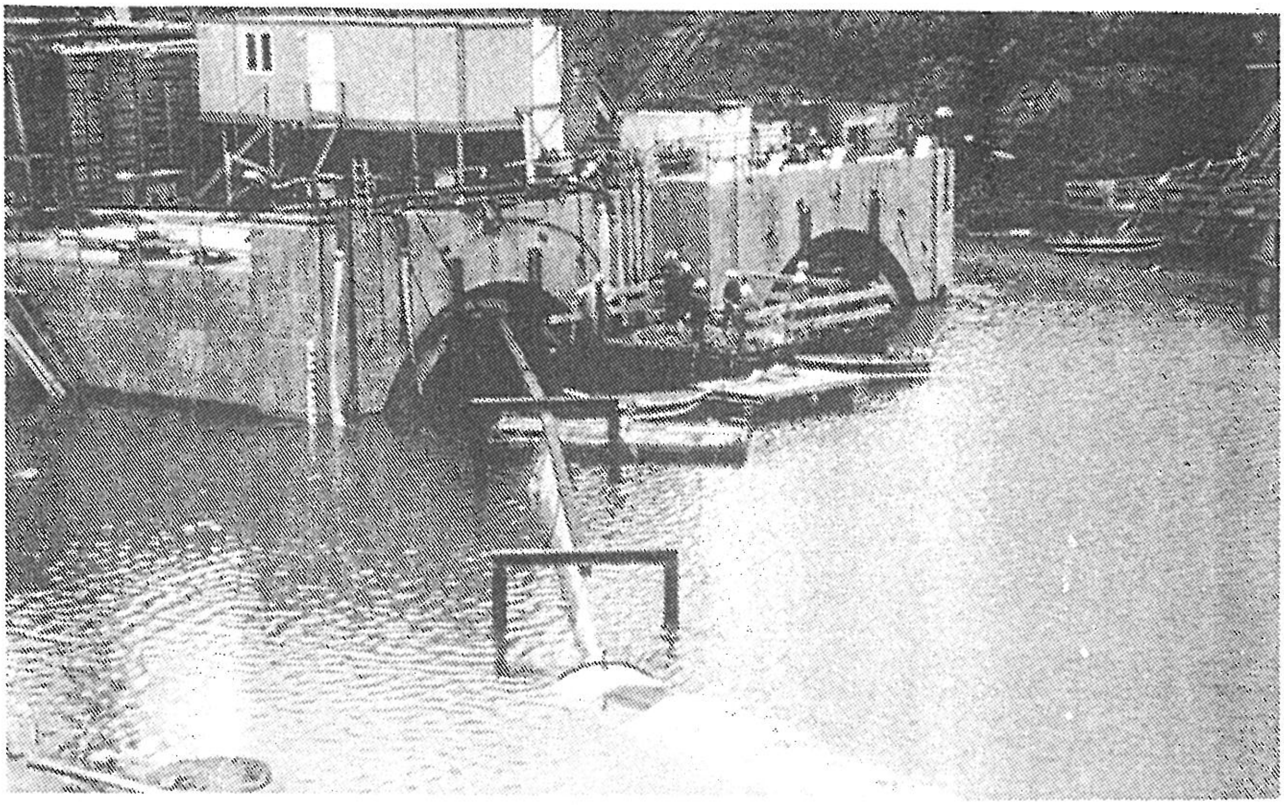


Fig. 23 Jutlandia incident with inundation of the tunnels and ramp on Sprogo 14 Oct. 1991

No doubt the acronym *MOSES*, *Method of Obtaining Safety by Emptying Storebælt*, was coined with some knowledge of Eckerberg's wonderful painting seen in Figure 24.

Project *MOSES* entailed large scale dewatering below the seabed with the objective to improve the safety of mining operations.

The immediate aim was to reduce the pore water pressure at the TBM axis by 3-4 bar in order to arrive at a nominal pore pressure of 3 bar at the tunnel axis, compatible with the use of compressed air for manned intervention in the TBM head. Furthermore it was important to improve soil stability at the cutter head and as a side effect to reduce the need for local dewatering at cross passages (more information may be found in Biggart et al, 1993; Odgård et al, 1993).

5.1 *History of project MOSES*

The first hint in the direction of the *MOSES* project was found when an onshore dewatering scheme was started in 1989 for the tunnel ramp area on New Sprogø. The horizontal extent of the dewatering was considerable as first experienced by the caretaker at Sprogø, as the sweet water well here dried out in October/November of 1989.

During the 1990-91 boring campaign for the East Bridge, it was confirmed that the extent of the Sprogø dewatering was large enough to produce down drags of the order 2-10 m in boreholes for anchor block west and approach piers some 1-3 km away from the ramp area.

In effect the Sprogø dewatering with a yield of 1100-1300 m^3/h produced down drags of 25-34 m in the ramp area with a radius of influence of 3-4 km. This constituted the factual background to set MOSES in motion.

During July to October 1992 a conceptual scheme for sub-sea dewatering was developed and in August '92 the Danish Geotechnical Institute conducted preliminary investigations sinking a number of boreholes and conducting trial pumping for the *MOSES* project on Halsskov and Sprogo sides.This provided a platform for the initial test programme which allowed fundamental insight into the geologic/geotechnical/hydrological conditions pertinent to the success of the scheme.

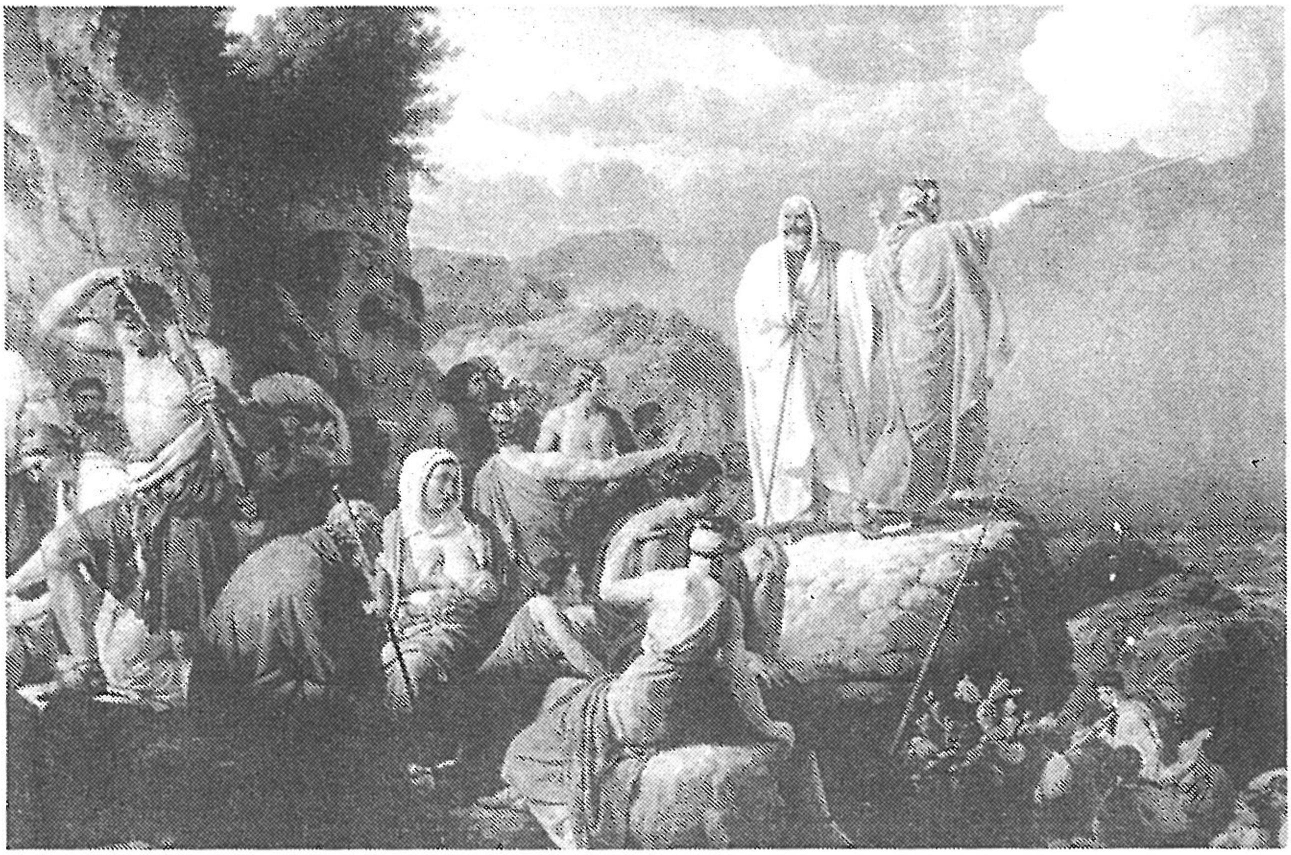


Fig. 24. Eckersberg's painting of Moses at the Red Sea

The trials showed that the sand layers dewatered for the Halsskov ramp were hydraulically connected to the marl. Hence, it was inferred that dewatering of the marl would cause a reverse effect and drain the overlying tills.

This was helped by the presence of high conductivity zones in the top of the otherwise low conductivity marl.

5.2 Operation of *MOSES*

Using the information gathered during the initial trials a layout for *MOSES* was developed as shown in Figure 25.

Modelling the effect of superposition indicated an optimum well spacing of 100-150 m with wells placed adjacent to and in-between cross passages.

United Drilling Contractors UDC, a subsidiary of the Danish Geotechnical Institute, performed the drilling using mobile drilling rigs placed on 3 jack-up platforms (cf Figure 27). The extent of UDC's installation and marine spread was as follows:

- ▼ 49 pumping wells (12" - 16")
  - a total of 2900 m drilled
  - yield range per well 15-200 m^3/h
  - total yield 45 million m^3
- ▼ 12 piezometer wells (8")
  - 650 m drilled

- ▼ Marine spread
  - 3 jack-up platforms with mobile drilling rigs
  - 3 supply and 2 diving ships
  - positioning accuracy of 0.3 m by DGPS.

The well heads were prototyped and manufactured by the Danish Geotechnical Institute.A schematic section of a typical marl well installation is shown in Figure 26. After completion of a well, installation of well head, submersible Grundfos pumps, pressure transducers, flow meter and discharge pipe the well was connected by seabed cables to MTG's set-up.

MTG was in charge of the daily running and monitoring of the total set-up:

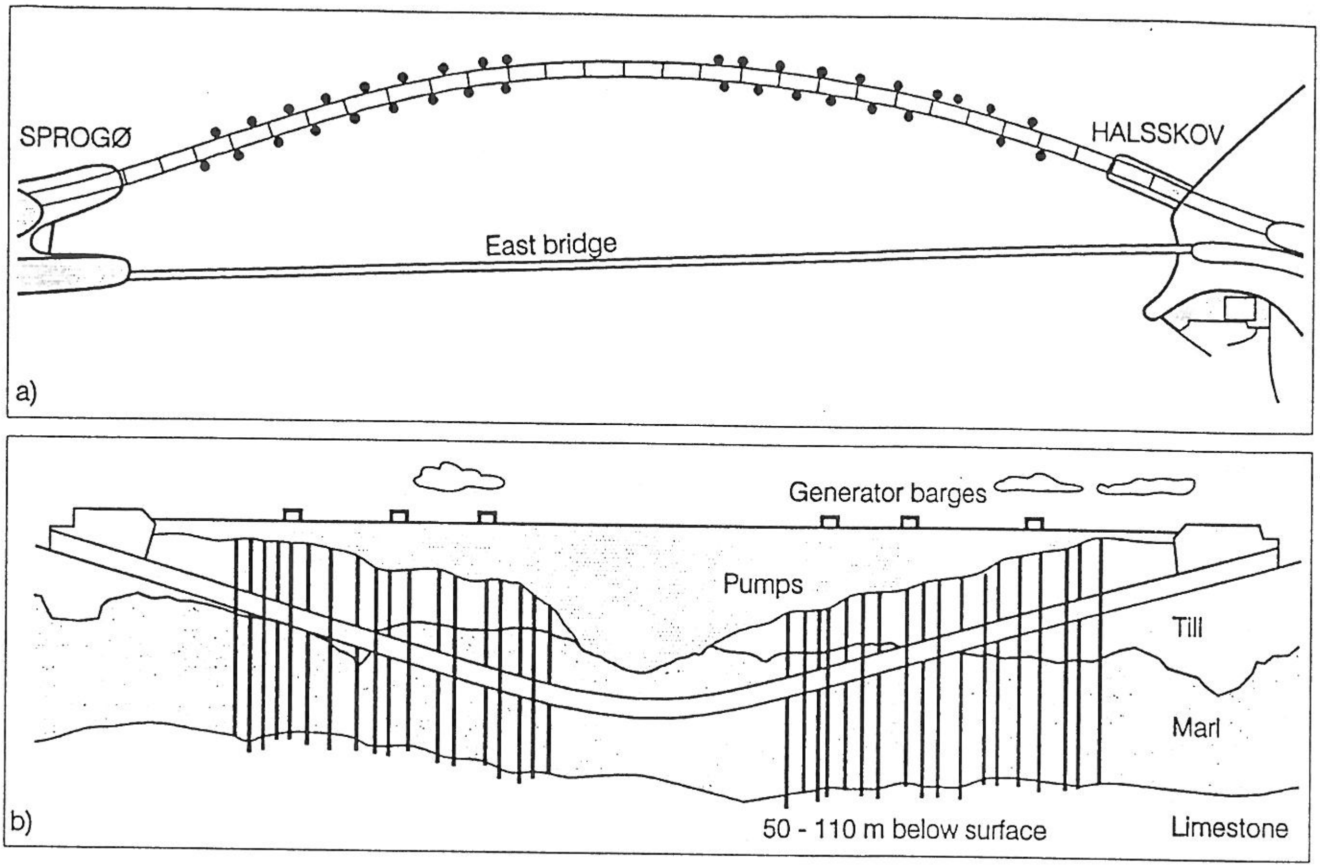


Fig. 25. Schematic layout of MOSES project. (a) plan of well positions; (b) alignment cross section

- ▼ 6 generator barges (one spare barge kept ready in Halsskov harbour)
  - power input 12 times 455 kW
  - total power 2 MW
  - seabed cable length 60 km
- ▼ Radio-telemetry link to Halsskov

The master control system in Halsskov was fitted with extensive alarm facilities for rapid response in case of break-downs or abnormal readings.

5.3 *Special challenges close to the TBM*

To enhance the mining potential of TBM Dania, which had met adverse ground conditions at the clay till-marl interface, a subset of *MOSES* was initiated.

Six wells to a depth of approximately 70 m below sea bed were drilled very close to the Dania tunnel front as shown in Figure 27. The set-up closely resembled the original *MOSES* set-up. However, due to the proximity of the tunnel front special measures had to be taken.In view of the possible formation of depressions over the tunnel axis UDC required guarantees for safe working conditions in connection with the operation of the jack-up.

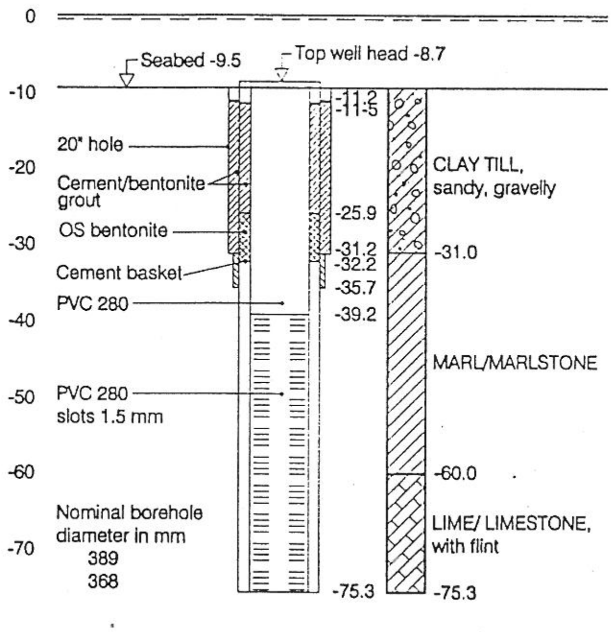


Fig. 26 Schematic cross section of typical marl well installation (levels and depths are metres)

Great care was taken to optimise leg positions and furthermore an alarm system, based on high precision satellite navigational data (DGPS) combined with heave-pitch and roll sensors, was mounted on the platform for on-line monitoring of any untoward movements of the platform.

The proximity of the tunnel front also required a precise 3D knowledge of the actual position of the boreholes in order to ensure that the drill string would not interfere with the cutter head when mining operations were resumed.

After placing of the casing from platform deck to sea bottom (cf Figure 27) and after completion of the well, the exact positions were established by inclinometer measurements.

Fig 28 shows that it is in fact possible to drill near-vertical holes even under these severe ground conditions with tills containing stones and boulders. However, Figure 28b also illustrates, that this is by no means a given thing, as obstacles may certainly deflect the drill string in the plane at a given depth. The inclinometer measurements, however, once more bore out the value of the seismic data collected for the project. The deflections of the boreholes all occurred at seismically determined soil boundaries and coincided with notes of "stone" in the borehole logs.

5.4 *Effects of MOSES*

The original design objective, to reduce the water pressure to a nominal 3 bar at the tunnel axis, was by and large achieved by the *MOSES* project together with the secondary objective, to reduce the water inflow indicated by the site investigations and *MOSES* trials.

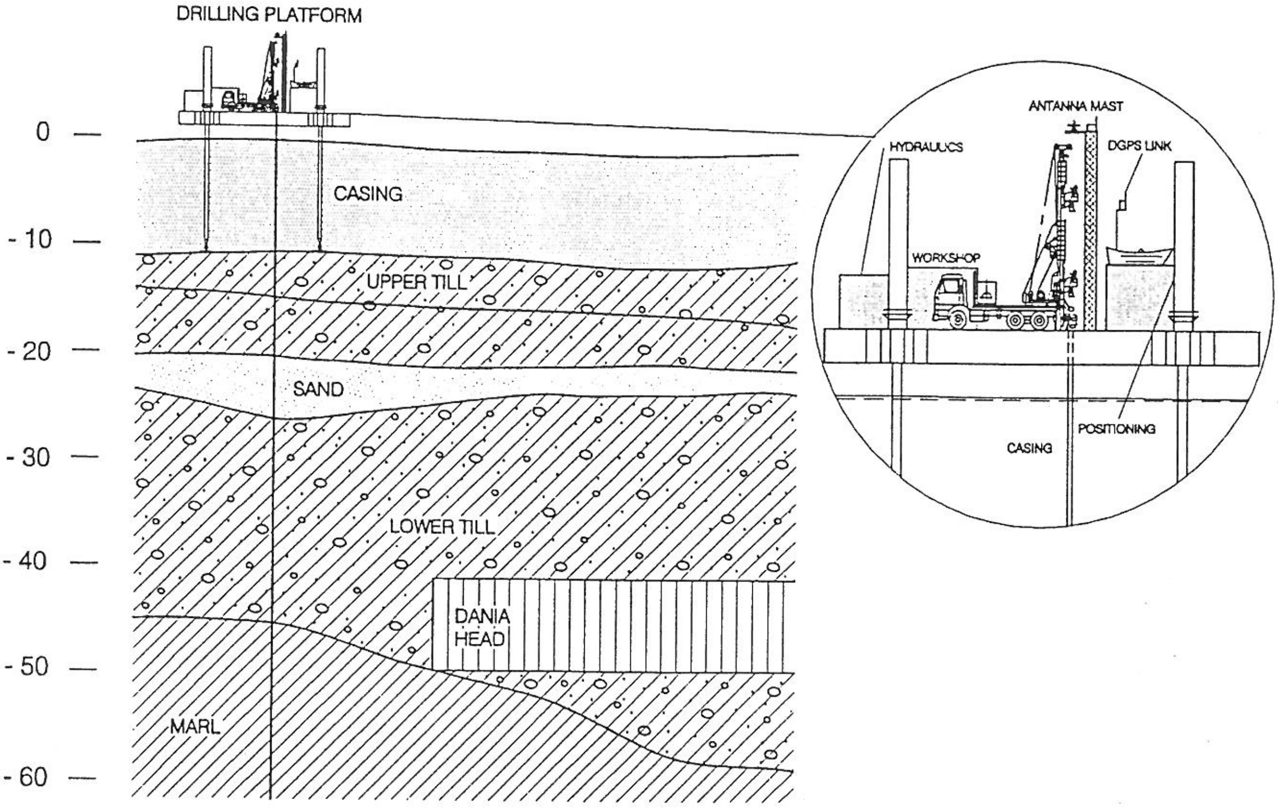


Fig. 27. Schematic set-up of MOSES boring close to TBM tunnel front

The lowered groundwater pressure has furthermore improved the conditions for site investigations and reduced the extent of ground treatment required for the construction of the cross passages. Even under the Nadir a significant reduction in pore pressure was achieved.

With project *MOSES* tunneling was resumed after the 1991 incident with rates of up to 134 m/week.

The *MOSES* project, at an estimated total cost of 200 million DKK, was the single largest job ever undertaken by the Danish Geotechnical Institute. During its 30 months of operation some 45 million m^3 of water was extracted from the Selandian marl and the glacial tills.

The costs may seem high. But the added safety and overall feasibility of actually completing the tunnels by reducing delays which might have occurred without *MOSES* in all likelihood recovered the costs many times over.

Project *MOSES* demonstrated that when client, contractor and designer meet in a co-operative approach then new borderline techniques can be successfully adopted for special applications

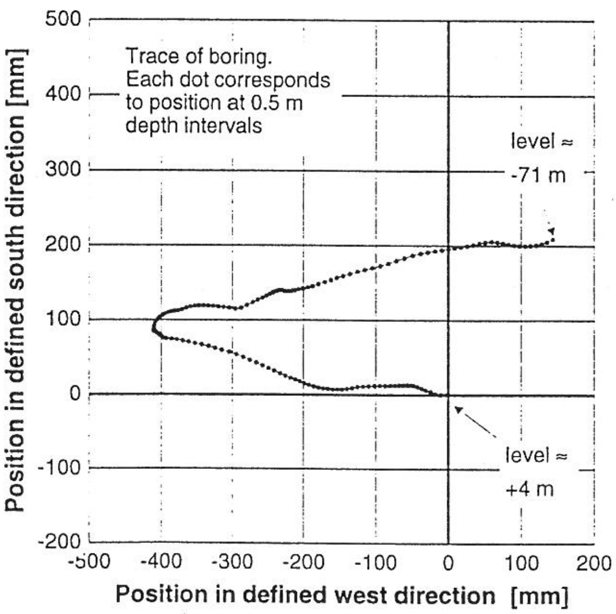
6. SLIDING TESTS

A sliding type failure was found to be important for the design of the East Bridge anchor block foundations subjected to significant cable pull and for the West Bridge piers subjected to impact forces from ice and ship collisions. In both cases the potential sliding would take place in the clay till below the foundations.

The sliding resistance was investigated experimentally by large scale field sliding tests and more extensively in the laboratory in a purpose-built Large Sliding Box.

In a multi-factor test programme the effects from ageing, over consolidation, pre-shearing, displacement rate and loading rate were investigated for Storebælt clay till.

This Section presents the test programme, the basic results and tentative correlations for the undrained sliding resistance. The description will focus on the more extensive investigations for the anchor blocks, but at the same time draw on the total amount of information from the West and East Bridges.



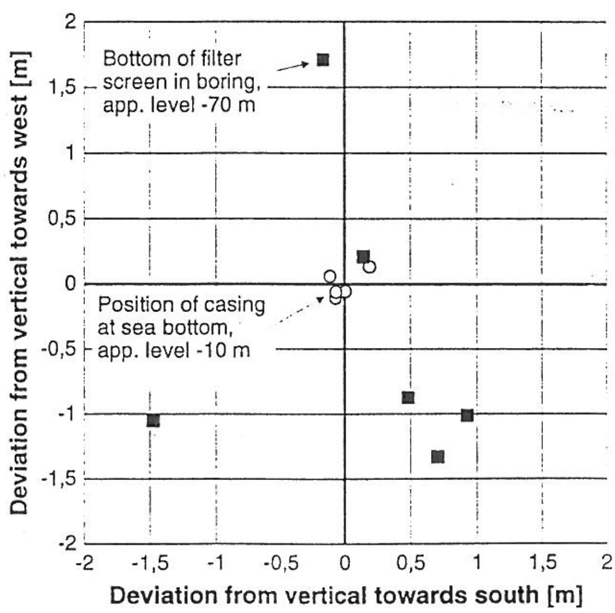


Fig. 28 Results of inclinometer measurements. (a) trace in plan of one of the boreholes from sea bottom to the end of the boring (each dot corresponds to an increase in depth of 0.5 m); (b) plan of borehole positions at seabed and at bottom of boring

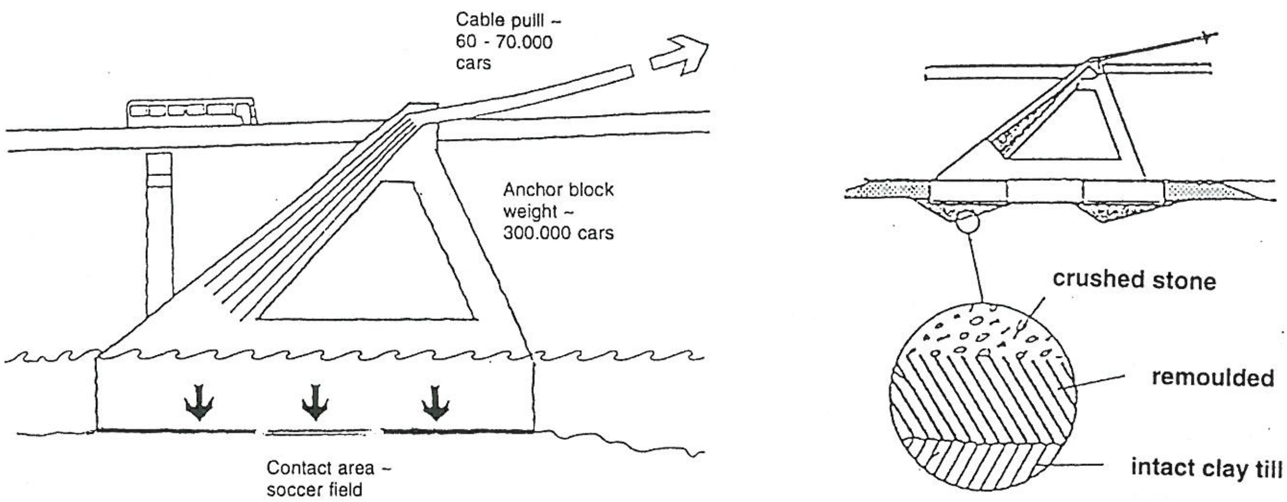


Fig. 29 Sliding problem for East Bridge anchor blocks (a) Anchor block loading; (b) Sliding interface between crushed stone bed and underlying clay till

More detailed descriptions may be found in Hansen et al (1991) for the West Bridge tests and in Steenfelt (1993) for overall testing.

6.1 *Sliding resistance problem*

The sliding problem may be exemplified by the East Bridge anchor blocks, as shown in Figure 29, where the cable pull and the dead weight of the anchor block structure result in a load ratio *H/V* at the interface between the foundation and the clay till.

In the experimental programme the resistance and loads are not factored by partial factors of safety, and hence *H/V* is also the resistance ratio, where *H* and *V* are the resistance components parallel with and perpendicular to the foundation/clay till interface. The interface is shown schematically in Figure 29b, where the loads from the concrete substructure are transmitted through a crushed stone wedge to the clay till.

Due to excavation and placing of crushed stone the clay till in the interface will inevitably be disturbed and remoulded. As it proved difficult to quantify the degree of disturbance with any confidence the investigations focused on completely remoulded clay till, to provide a safe, lower boundary, and intact clay till.

6.2 *Test types and testing strategy*

The problem of sliding resistance for clay till was specifically addressed by testing at different scales and different interfaces:

- 28 field sliding tests using 1 by 2 m^2 concrete blocks,
- more than 70 Large Sliding Box tests on 400 mm cylindrical clay till specimens in the laboratory and
- 20 conventional direct shear box tests on 100 mm square and 30 mm high clay till specimens in the laboratory.

Due to the cost and complexity of field testing most of the parameter and correlation studies were performed in the laboratory, where repeatable and controlled test conditions may be provided.

The field tests were then used as bench mark tests to check the correlations and to provide the means for safe extrapolation to prototype scale. The cost dictated strategy is borne out by Figure 30, where the cost of testing is compared to the sliding area in question.

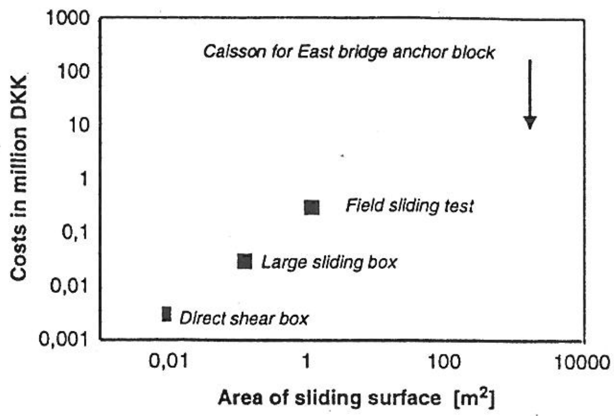


Fig. 30 Cost of tests versus sliding area

From a cost-benefit point of view the Large Sliding Box tests were considered optimal.

6.3 *Storebælt clay till*

The clay till used for the experimental investigations is Storebælt Clay Till, a clay till which is so macroscopically striking that it can be recognised wherever it occurs in borehole profiles. It is a very dark grey, fine sandy clayey till which contains mm-sized "augens" of calcareous material. The geological history of the clay till is described by Foged et al (1995).

Due to the very extensive investigations in the Storebælt area, the clay till is differentiated in a number of distinct types. They are clay tills of low plasticity exhibiting varying degrees of overconsolidation.

The grain size distributions vary slightly, but on average the clay content is 18% (silt content 32%, sand content 45%, gravel and boulders constitute the remainder).

Table 4. Average classification parameters for Storebælt Clay Till used in LSB-tests

Soil parameter	Symbol	Average value
Natural water content	w	11 %
Liquid limit	w_L	16 %
Plastic limit	w_P	10 %
Plasticity index	I_P	6 %
Unit weight of solids	γ_s	26.9 kN/m^3
Lime content	$CaCO_3$	21 %

The clay minerals (≤ 10%) are dominated by expandable clay minerals and illites.

The classification parameters corresponding to anchor block conditions are summarised in Table 4.

6.4 *Large Sliding Box tests*

An element of the interface between a concrete structure, with or without a stone layer, and underlying clay till was simulated in a purpose-built Large Sliding Box, see Figures 31, 32 (cf Bak & Steenfelt 1992).

Quite literally, the interface problem was mirrored, as shown in Figure 31. A cylindrical sample of clay till with a diameter of 400 mm and a height of 100 mm is forced to slide against a horizontal surface of steel or gravel-coated steel.

Two types of clay till interfaces were used in the model set-up corresponding to intermediate and maximum disturbance:

- *Disturbed*, where the intact clay till at natural water content is broken down to gravel size ($d \approx 30$ mm), and
- *Remoulded*, where the clay till is completely remoulded (at $w \gtrsim w_L$).

Thus, the clay till conditions closely resemble the non-intact conditions for the field sliding tests (cf Hansen et al 1991).

In order to allow for sliding only part of the cross sectional area was gravel coated. Coating area ratios of 77, 56 and 0% were used to facilitate extrapolation to 100% gravel coating.

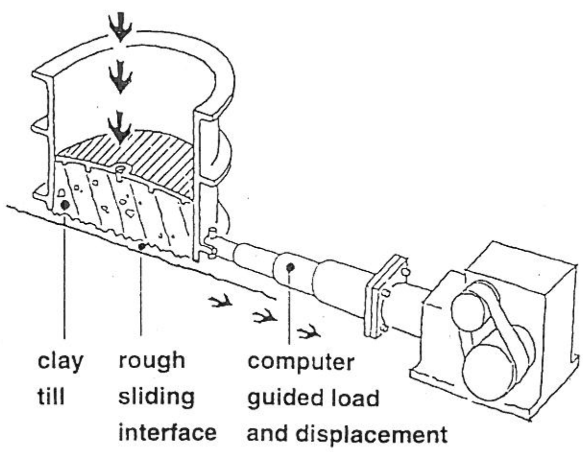


Fig. 31 Principle of Large Sliding Box

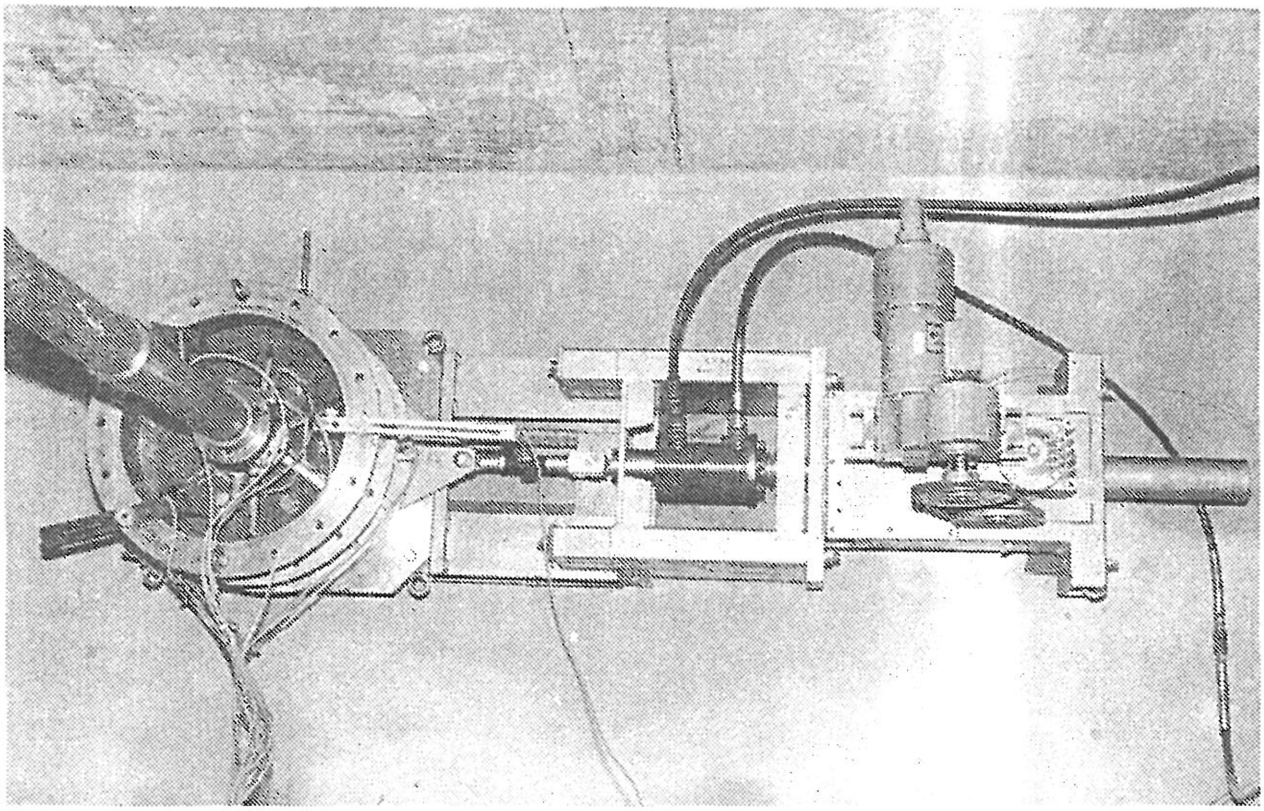


Fig. 32 Overall view of Large Sliding Box (LSB)

The specimen is loaded vertically by a load-controlled hydraulic jack The horizontal sliding can be achieved either by a spindle with electronic displacement control or by a pressure or displacement controlled hydraulic jack, or by both systems simultaneously.

The displacements of the clay till specimen in vertical and horizontal directions together with the vertical and horizontal loads are recorded by an automatic data acquisition system. The set-up allows displacement rates from 0.1 to 2000 mm/h and horizontal shear stress rates from 1 to 1000 kPa/h. However, the upper limit of the shear stress rate is intimately linked to the accompanying displacement rate.

6.5 *Field sliding tests*

The basic principle of field sliding testing is shown in Figure 33.

A dead-man anchor is obtained by vertical loading of a large concrete block by eight ground anchors. This is used as a reaction block for horizontal loading of the concrete test block of 1 by 2 m^2 area using a computer controlled load/displacement system based on a servo motor and a hydraulic jack (cf Figure 34). The test block is loaded vertically by four hydraulic jacks using reactions from four ground anchors.

Figures 34 and 35 show that the 300 mm casing around the top 10 metres of the ground anchors allows unrestricted horizontal movement of the test block.

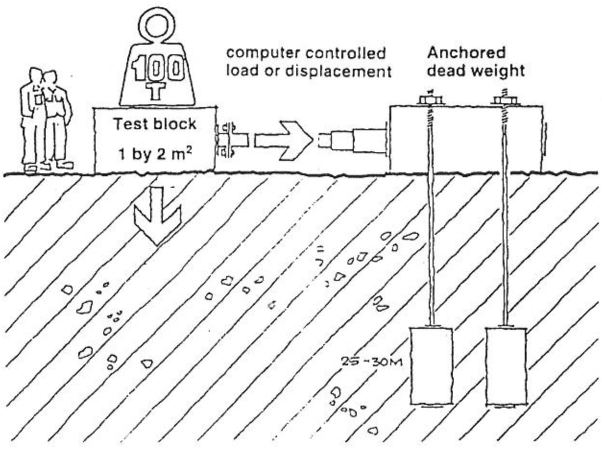


Fig. 33 Principle of field sliding tests

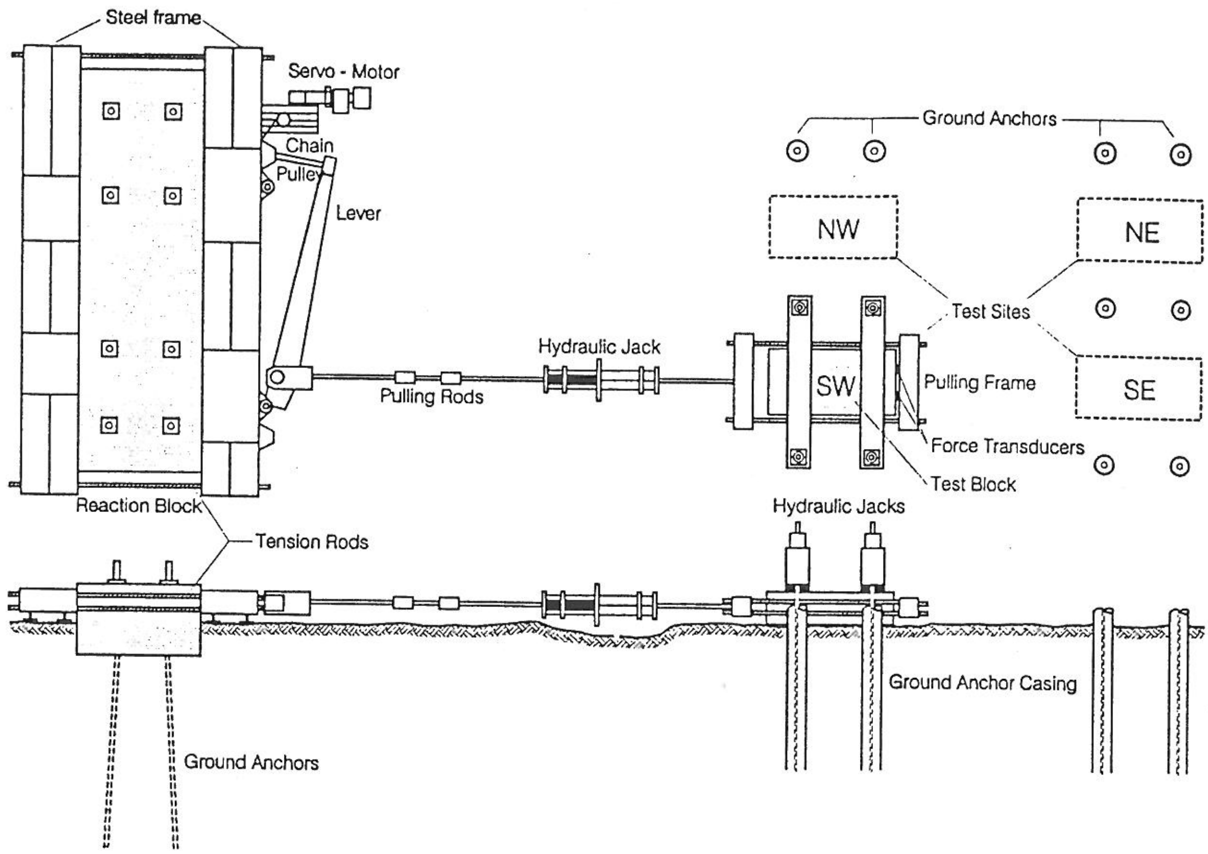


Fig. 34 Principle of field test set-up (Horizontal loading system is in a neighbouring tent)

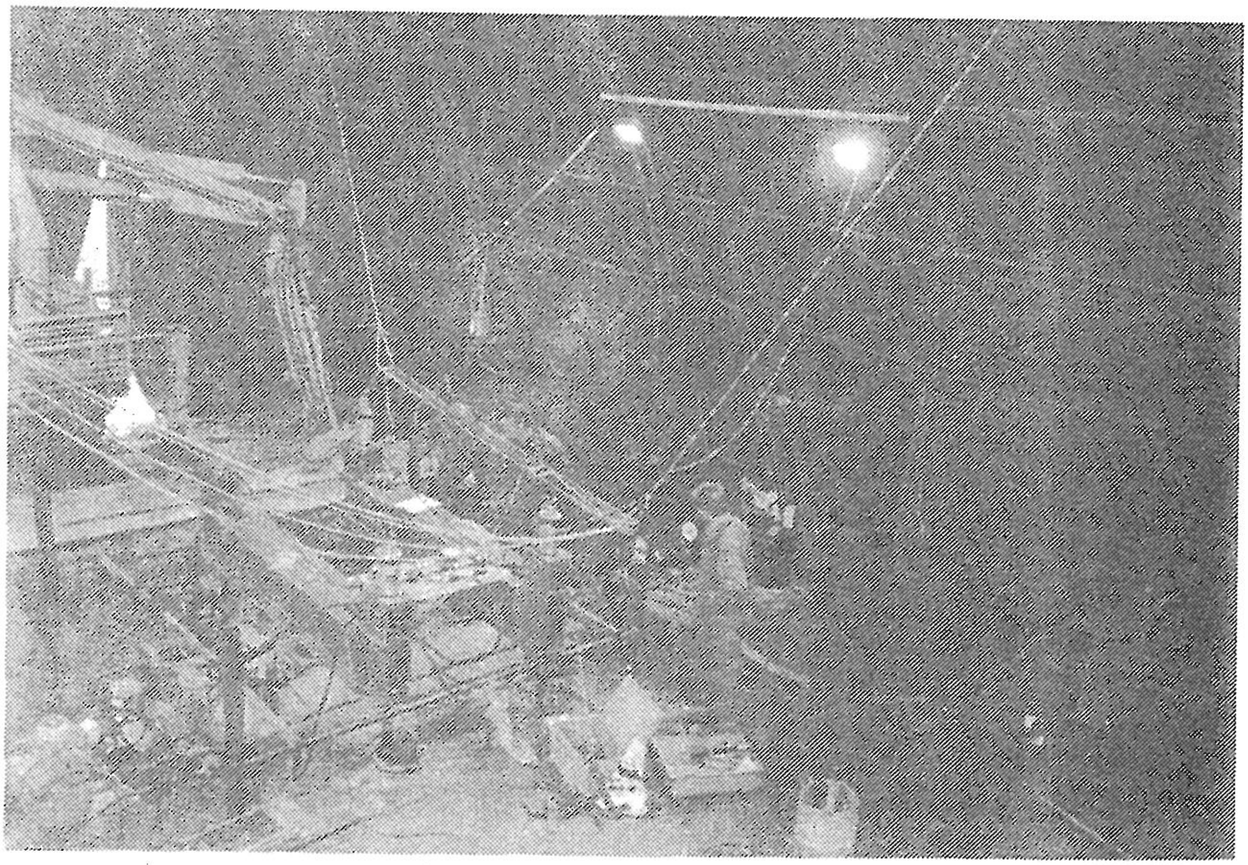


Fig. 35 Set-up of field sliding tests in main tent with control and office facilities

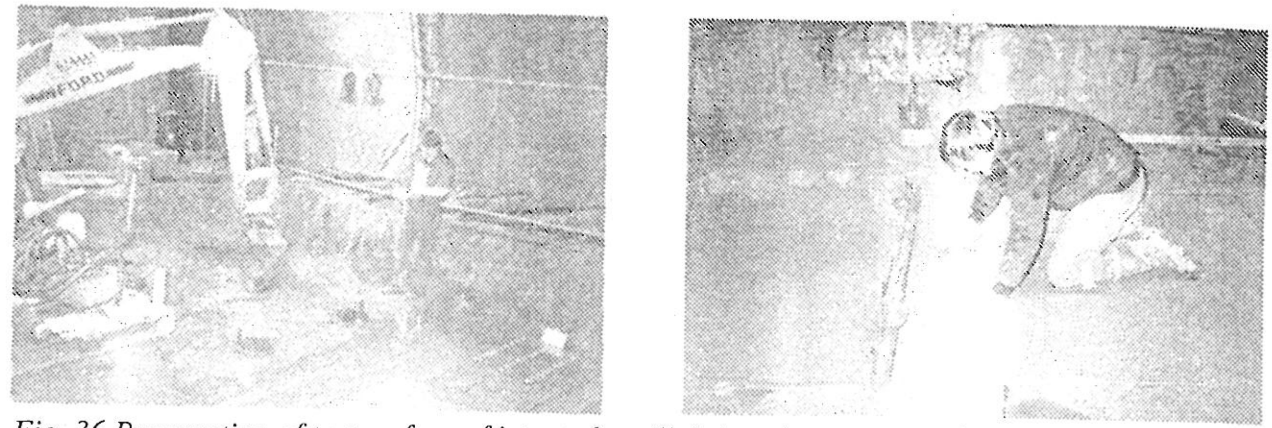


Fig. 36 Preparation of test surface of intact clay till. (a) grab-excavator; (b) finish by hand tools

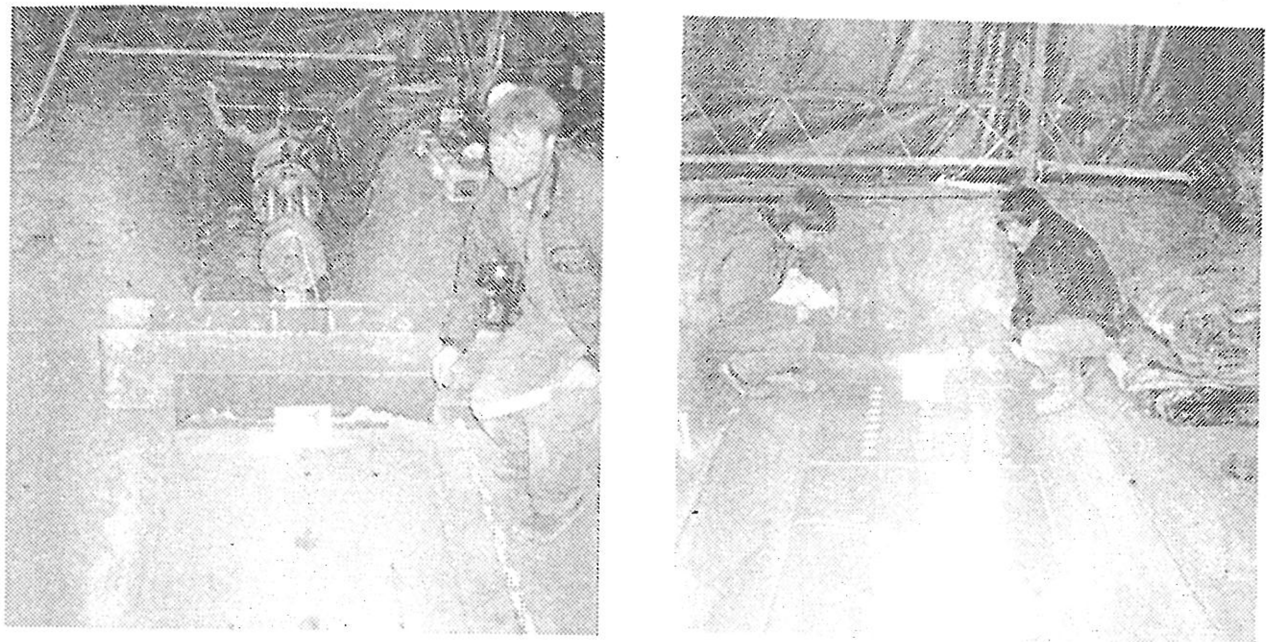


Fig. 37 Monitoring of displacements in interface by markers (coloured sand in grid pattern)

The test site was situated close to Storebælt's Exhibition Centre in Halsskov. As seen in Figure 35 the test set-up and test site were protected by a large tent for climatic reasons.

In all cases the top soil was removed to a depth where intact Storebælt clay till was present. Then a horizontal plane surface is very carefully prepared, as shown in Figure 36, using grab-dredging, shovel and finally hand tools.

The displacements in the potentially affected clay till zone were monitored by initially vertical tell-tale holes drilled into the clay till and filled with markers at the top of the clay till (Figure 37) and washers or coloured clay (Figure 38, showing special transparent sheets for easy recording of failure patterns).

Tests on remoulded clay till were achieved by placing a 200 mm thick layer of clay till slurry in a basin excavated in the intact clay till.

Fig. 38 Coloured clay marker exposed after test (on the 10 mm grid the sliding surface is seen some 50-60 mm below the surface)

6.6 *Results of sliding tests*

One of the notable results of the sliding tests was the very clear influence from displacement rate, $\dot{\delta}$, on the shearing resistance *H/V*. The combined results for remoulded clay till shown in Figure 39 suggest a minimum resistance at $\dot{\delta} \approx 120$ mm/h. "Fortuitously" this is the standard displacement rate applied in shear box testing in the laboratory!

For decreasing displacement rates a near-drained sliding failure is reached at $\dot{\delta} \approx 0.5$ mm/h in the LSB-tests, whereas the field tests show a transition to drained bearing capacity mode of failure (transition from element to model test?).

In order to compare displacement and load controlled tests in the LSB, the time to failure has been used in Figure 40 as controlling parameter. The same trough-like behaviour is recognised, but it is also clear that the transition from drained to most adverse condition is much more rapid for the load controlled tests. Furthermore, the influence of increasing loading rate is not as pronounced (very flat minimum).

The difference in behaviour between displacement and load controlled tests can also be seen in the load-displacement curves (Figure 41).

Figures 39, 40 amply demonstrate that the drained and undrained conditions are not clearly defined and alternative descriptions, particularly for the so-called *undrained state* may be needed. This state not only depends on the rate of pore pressure dissipation, but creep and degrees of freedom in the test (confinement) also play an important role.

Early in the investigation it became clear that the ageing time, or creep, played a very important role. Figure 42 shows the effect of the ageing time t_a - here defined as the time from initiation of the final, vertical consolidation step to start of sliding - on the resistance ratio *H/V*.

Tentatively a cautious approximation of the increase in resistance ratio with t_a, may be described by:

$$(H/V)_{nc} = 0.19 + 0.06\log_{10}(1 + t_a / 1day) \quad (1)$$

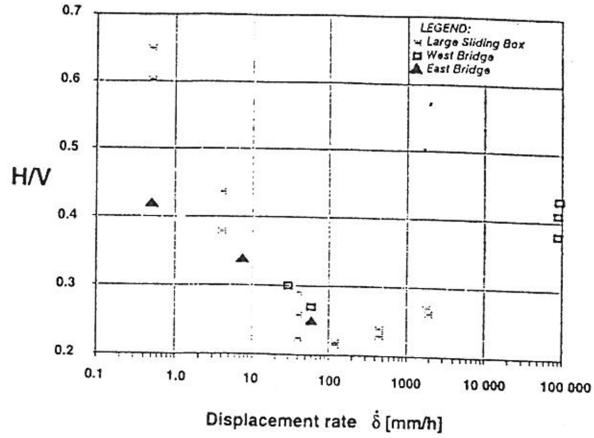


Fig. 39 Resistance ratio versus displacement rate for remoulded clay till (corrected for t_a)

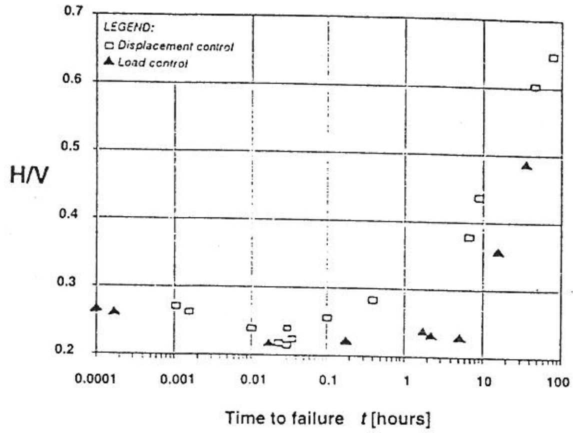


Fig. 40 Resistance ratio versus time to failure in displacement and load controlled tests (LSB)

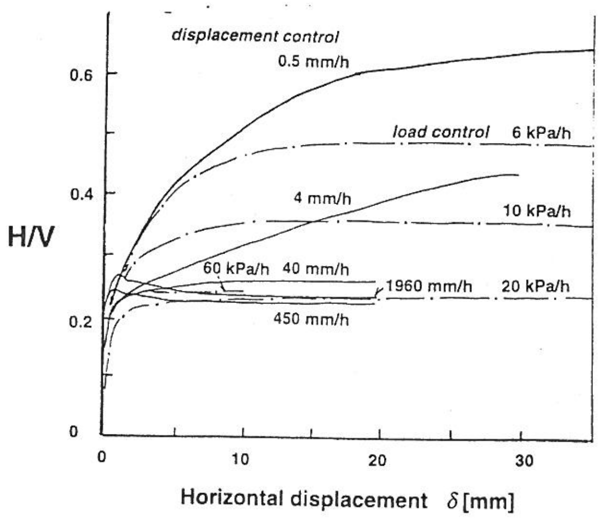


Fig. 41 Load-displacement curves for LSB tests on remoulded clay till

This indicates that one must be very careful not to postpone testing over weekends or holidays without proper attention to the effect on test results. Eq (1) was used to harmonise data corresponding to different ageing times when other parameters were examined (compare Figure 39).

For the East Bridge anchor blocks a permanent shear stress will be slowly applied in the soil-structure interface due to the build up of cable pull during construction.

This was simulated by a series of *LSB* tests where a sub-failure shear stress was applied after completion of primary consolidation of the final vertical consolidation step. Figure 43 shows the results with data adjusted to $t_a = 72$ h according to Eq (1). A very consistent trend emerges with a cautious limit of:

$$H/V = 0.22 + 0.75 H_0/V - 0.05 \tan^{-1}(15 H_0/V) \qquad (2)$$

where $H_0/V = \tau_0/\sigma'_c$.

The increase in shear resistance is very significant as the ubiquetous effects, such as ageing, have been subtracted already.

Finally, the effect from overconsolidation was studied in *LSB* tests where a number of specimens were normally consolidated at σ'_c = 600 kPa and then allowed to swell back to different values of σ'_{red}.

For triaxial testing the *SHANSEP* approach was found to be relevant for presentation of undrained strength data for Storebælt clay till (Steenfelt & Foged, 1992; Steenfelt & Sørensen, 1995) corresponding to

$$(c_u/\sigma'_{red})_{oc} = (c_u/\sigma'_c)_{nc} OCR^{\Lambda} = 0.42 \ OCR^{0.85} \qquad (3)$$

Using $\dot{\delta} \approx 120$ mm/h and assuming this to correspond to undrained failure the same type of approach for the sliding problem leads to

$$(H/V)_{oc} = (H/V)_{nc} OCR^{\Lambda} = 0.219 \ OCR^{0.73} \qquad (4)$$

as shown in Figure 44.

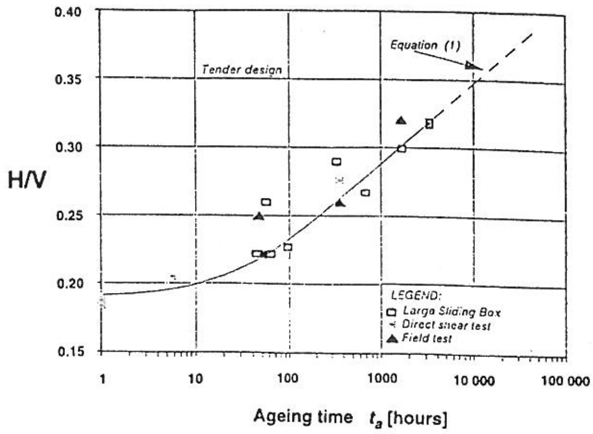


Fig. 42 Resistance ratio H/V versus ageing time t_a ($\dot{\delta} \approx 60$-120 mm/h)

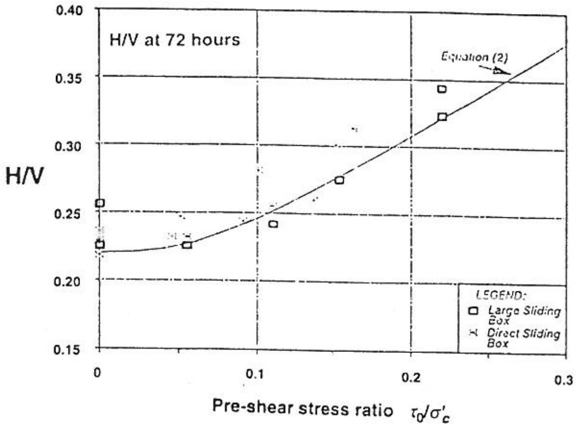


Fig. 43 Resistance ratio H/V versus pre-shear stress ratio τ_0/σ'_c

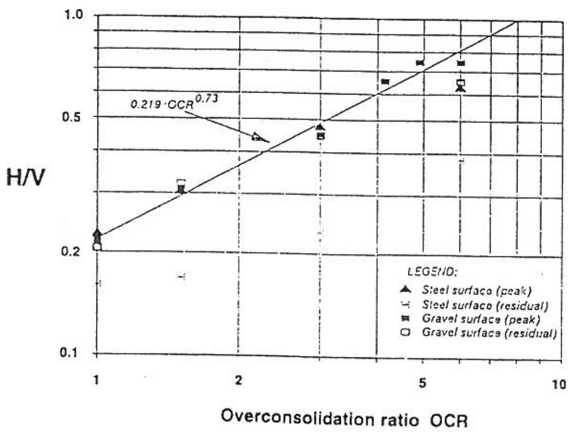


Fig. 44 Resistance ratio H/V versus overconsolidation ratio OCR (σ'_{nc} = 600 kPa)

6.7 *Recommendations and conclusions*

The sliding resistance parameters to be used in design were found to be intimately linked with:

- actual soil conditions in the excavated surface prior to positioning the structure,
- the actual loading history for the structure,
- the safety philosophy adopted in design, and the
- engineering judgement in assessment of deviations from the basis for established correlations.

Combining eqns (1), (2) and (4), it was found that the sliding resistance ratio for a rough interface to remoulded Clay Till 0-1 in normal or overconsolidated state could be described as:

$$(H/V) = \left[0.19 + 0.06\log_{10}\left(1+\frac{t_a}{1\ day}\right) + 0.75\ H_0/V_0 - 0.05\ \tan^{-1}(15\ H_0/V_0)\right] OCR^{0.73} \quad (5)$$

Eq (5) assumes that the static load on a horizontal surface, with vertical and horizontal components V_0 and H_0, has acted t_a days prior to the increase in loading ratio which might take the structure to failure. Further, the pre-shear stress has been in the same direction as the shear stress increase and the pre-shear stress ratio has been H_0/V_0.

The effects of other loading histories may also be evaluated based on the data base. However, each case would require a specific analysis in order to assess the validity of application of the established trends.

The trends in Sec 6.6 were in general unequivocal. The fact that sliding resistance was checked in both the laboratory and the field at three different scales lends credence to the possibility of extrapolation to prototype scale.

Using eq. (5) for the assumed loading history of the anchor blocks an extrapolation as shown in Figure 45 results.

In the conceptual design a resistance ratio of $H/V = 0.35$ was required. This can obviously be achieved, but closer scrutiny of combined failure mechanisms and probabilistic safety studies required that the crushed stone beds shown in Figure 29 were maintained (cf Sørensen et al, 1993).

Possible failure mechanisms in the crushed stone beds were examined by small scale conceptual model tests in the laboratory (Steenfelt et al, 1994).

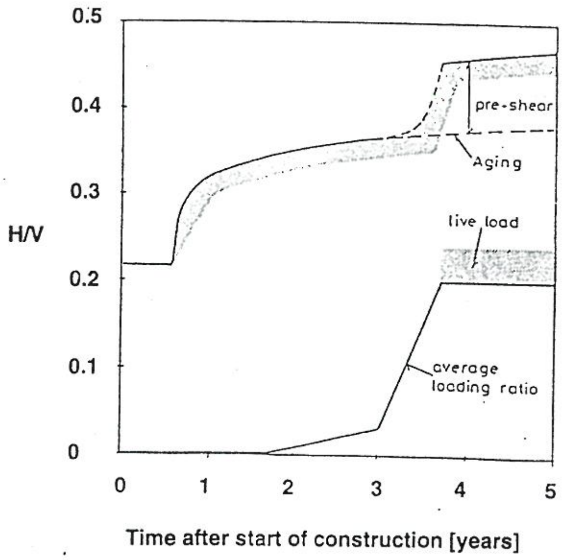


Fig. 45 Extrapolation of sliding resistance based on eq. (5) compared with action ratio for East Bridge anchor blocks

7. SPIN-OFF AND LESSONS LEARNED

During the completion of the Storebælt Link project, the geotechnical profession was met with a series of challenges and well established practices and techniques had to be reviewed.

The Danish well-winnowed tradition of interplay - in the broadest sense of the Conference topic, i.e. in-between very different professions - was proved extremely beneficial.

It is crucial for the success of such a project, that a transparent system for evaluated, stored and retrieved geotechnical, geological, hydrological and geophysical data is established and maintained.

Special care must be exercised when mixing old and new data and data from different sources and cultures. In the light of the growing internationalisation the understanding of and respect for local and global experience and soil conditions was a challenge for expatriates, and Danes alike, involved in the project.

For the Danish geotechnical community at large the Storebælt project was the stepping stone to a number of initiatives. Apart from offshore jobs, this was the first project where strict quality assurance requirements were enforced - for better and for worse. This has had the beneficial effect that the involved companies are now much better equipped to work in the international market.

In general, improved procedures were developed for understanding and handling soil types otherwise considered well known.

For the Danish Geotechnical Institute the project instigated construction of new purpose-built field and laboratory equipment, such as

- the offshore CPT rig SCOPE,
- the Large Sliding Box,
- an automated direct shear box, which may be transferred into a constant rate of strain oedometer,
- a large scale, rugged triaxial set-up for testing of coarse and heterogeneous materials with 500 mm diameter specimens, (cf Steenfelt & Foged, 1994).

The project also amply demonstrated that Murphy's laws are still valid. There were a number of Gordian knots and disappointments but also break-throughs as shown in Figure 47, where His Royal Highness Prince Joachim of Denmark inaugurates the break-through of the southern tunnel tube in October 1994.

However, the overriding message from the authors - after many years of involvement in the project - is that geotechnics are still exciting.

We are looking forward to being able to cross Storebælt on its first permanent fixed link and enjoy the scenic beauty of this civil engineering project (cf Figures 46 and 48).

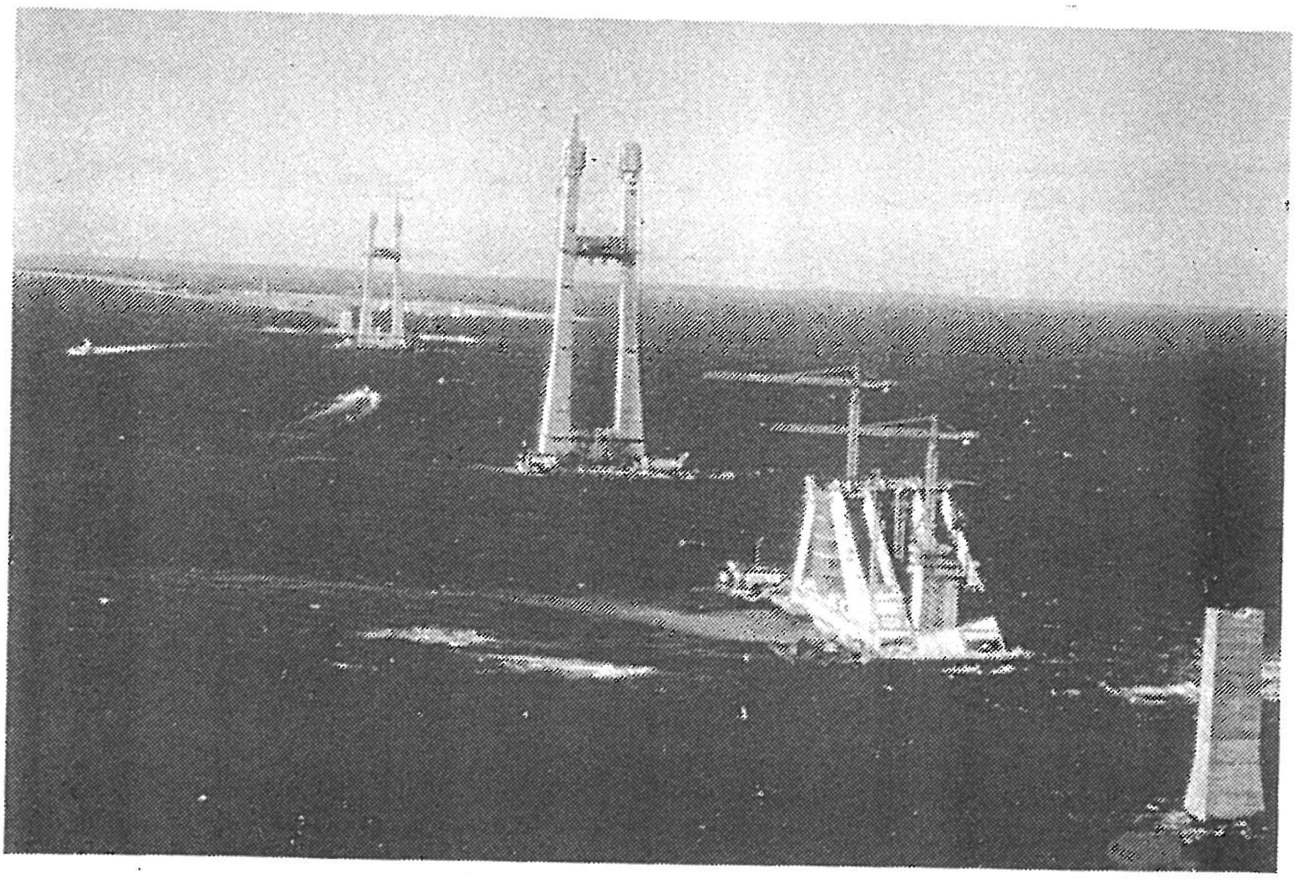


Fig. 46 The East Bridge set against Sprogø in early 1995.

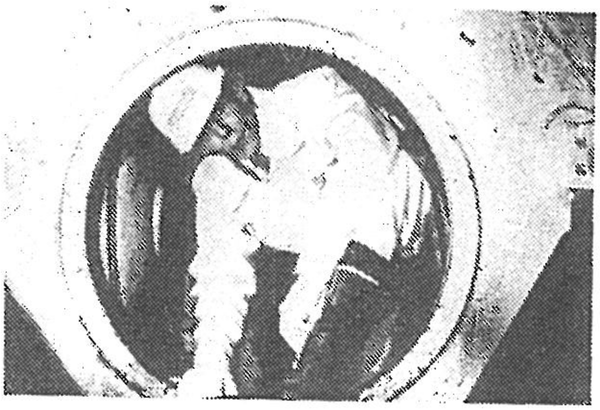


Fig. 47 The break-through of the southern tunnel tube October 15, 1994

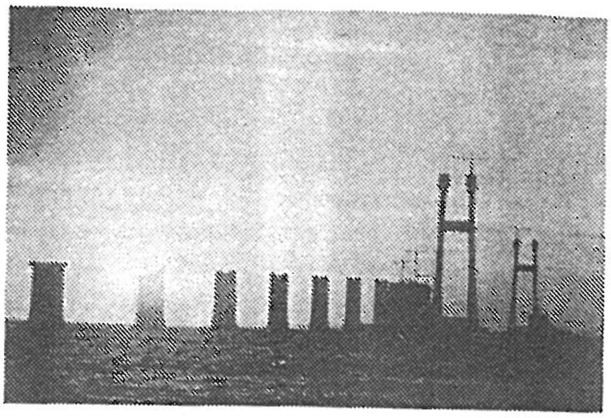


Fig. 48 The sun setting behind the East Bridge pylons in early 1995

8. ACKNOWLEDGEMENTS

The authors gratefully acknowledge permission by A/S Storebæltsforbindelsen and the Danish Geotechnical Institute to publish the paper.

It goes without saying that a large number of colleagues have contributed directly, by personal communication, or indirectly by reports and papers submitted concerning the project. We thank them all very warmly for their assistance and encouragement. A special thank to Aage Hansen for constructive comments to the paper.

It has been a privilege to be involved in the Storebælt Link project.

The help by *Storebælt BilledArkiv* in providing excellent slides for the Key Note Address is gratefully acknowledged (Figures 2, 9, 13-14, 16-18, 20-23 and 46-48 in the written version).

The remaining photos are taken by the first author.

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